BEHAVIOR - BASED SIMULATION APPROACH FOR THE CAPACITY AND TRAFFIC OPERATIONAL CHARACTERISTIC STUDY OF FOUR-WAY-STOP-CONTROLLED INTERSECTIONS

By

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Behavior-Based Simulation Approach for the Capacity and Traffic Operational Characteristic Study of Four-Way-Stop-Controlled Intersections

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ABSTRACT

The objective of the proposed research is to study the traffic operational characteristics at four-way-stop-controlled (FWSC) intersections with single -lane approaches. Observational data were collected at six FWSC intersections in Lawrence, Kansas. Then, the necessary traffic data were extracted from the videotapes using data processing programs. With the collected data, a driver behavior based simulation approach was presented to estimate the 95th percentile queue length, control delay and service time. After calibration and validation, the simulation model was used to study characteristics of the queue length, control delay, and service time at FWSC intersections. Finally, intersection capacities were estimated under different traffic conditions.

The simulation results were compared with the HCM 2000 model and several other existing simulation models and theoretical calculation models. The following conclusions can be made:

(a) The capacity at FWSC intersections predicted by the simulation model in this study is generally higher than that based on the HCM 2000 model, but lower than those from several other studies.

(b) A move-up time longer than 2 sec. should be used based on the collected data and the simulation proved that a short move-up time may cause a longer service time if all other parameters are kept same. A 2 sec. move-up time is used in the HCM 2000 model and the service time calculated from the model is longer than the field service time.

(c) Based on the simulation model in this study, both right turn vehicles and left turn vehicles have significant impacts on the intersection capacities at FWSC intersections. On the contrary, the HCM 2000 model is not able to reflect the impacts of right / left turn traffic on FWSC intersection capacities.

(d) A FWSC intersection has the best performance under even volume splits on the two intersected streets with even directional distributions.

Finally, the limitations of the simulation model and recommendations for further research are discussed.

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CHAPTER 1 INTRODUCTION

1.1 RESEARCH BACKGROUND

Intersections play a significant role in roadway transportation systems. If not properly designed, they can limit the capacity of the entire traffic network. In addition, the capacity of an intersection is very important for traffic management and transportation planning. Therefore, the importance of finding an accurate estimation of intersection capacities cannot be overemphasized.

The four-way stop (FWSC) is a common intersection control widely used in the USA. It can be an interim measure for signal control or for improving safety. It is normally used when the volume of traffic on the intersecting roads is approximately equal. However, the research on capacities at FWSC intersections is relatively limited.

Capacity can be defined as the maximum hourly flow rate at which vehicles can reasonably be expected to traverse a point or an uniform section of a lane or a roadway during a given time period under prevailing roadway, traffic and control conditions. For FWSC intersections, the capacity is the maximum number of vehicles that can be reasonably expected to pass through the intersection during a given period of time under prevailing roadway, traffic and control conditions. Empirical methods and analytical methods are two frequently used techniques to determine the capacities of FWSC intersections.

Empirical models are based on statistical analysis of extensive field studies and often used when the real phenomenon is so complex that simple analytical equations could not be formulated. Besides, they are easy to use and understand. But, on the other hand, they are limited to certain traffic and geometric conditions, at which they are calibrated. Therefore, they may not have general applications. In addition, an empirical model cannot isolate the influence of one or a few factors affecting intersection capacity, such as driver behaviors.

Current FWSC theoretical models are based on either probability (2000 HCM model) or graphic theory (Addition-Conflict-Flow (ACF) model) with field data validation. Like empirical methods, they cannot consider all the influential factors. For example, the HCM 2000 considers only 5 cases at all-way-stop controlled (AWSC) intersections and neglects the effects of traffic turning movements. As a result, the model is not sensitive to turning movements. Moreover, it sometimes overestimates the conflicts among the traffic flows, but sometimes it underestimates the conflicts. The ACF model expanded 5 cases to 192 cases, according to the traffic conditions faced by the drivers on the subject approach. However, these cases were arbitrarily reduced to 5 categories, the same as the HCM 2000. Even though the ACF model improved the model's ability by reducing overestimating conflicts, it underestimated the conflicts to some extent.

At the same time, theoretical models are limited by the number of parameters they can incorporate into the formulas. Usually they "describe" simplified reality. It is difficult or even impossible to represent the interactions among the vehicle turning movements. Another shortcoming of the HCM 2000 and ACF models is that the queue length estimation is not provided.

Intersection capacity depends on a variety of factors including geometry conditions, traffic compositions on approaches, volumes, and vehicle interactions. Unlike signalized control, there is no priority assigned at AWSC intersections. Give- way- to- the- right and first- in- first-out rules are often taken as the assumptions for most theoretical models. In reality, traffic operations at AWSC intersections are more complicated. Driver behavior characteristics have an important influence on the capacity of an AWSC intersection. Considering that theoretical and empirical approaches cannot model driver behaviors, the use of these models to comprehensively represent the traffic characteristics at AWSC intersections in the real world is very difficult. Comparatively, simulation provides an efficient technique to estimate capacities at AWSC intersections because of its ability to simulate complicated traffic characteristics. Traffic simulation models use numerical techniques on a computer to create a description of traffic operations over an extended time. They can yield insight into how the variables interrelate and simulate different traffic conditions over time.

There are several existing simulation models to estimate capacities at AWSC intersections, including TEXAS, AWSIM, Chan's simulation model STOP-4 and Bristow's simulation model. The TEXAS model and Bristow's simulation model were developed in the1970's. Chan's simulation model was developed in the 1980's. All these three simulation models predicted that the capacity at an AWSC intersection is lower when the traffic volume is evenly distributed among all approaches compared to other traffic volume distributions. This prediction contradicts the findings from Wu, Richardson, and Hebert. A higher capacity under evenly distributed traffic demands was expected by The AWSIM simulation model, which was developed by Kyte et al. in 1996. However, The AWSIM model assumed that it took the same amount of time for left turn vehicles and right turn vehicles to pass through a intersection and did not consider the variance of different turning movement combinations. Another shortcoming of this model is that the capacity is based on queue delay and stop delay rather than control delay, which is used as the criterion of level of service at intersections. Accordingly, there is a need for an improved simulation package for AWSC intersections.

1.2 RESEARCH OBJECTIVES AND SCOPE

The primary objective of this research is to estimate the capacities at FWSC intersections based on driver behaviors and study the traffic operational characters including the 95th percentile queue length, average control delay, and service time. A simulation technique will be utilized to predict the capacities, control delays and queue lengths at FWSC intersections.

With the simulation model, capacities will be evaluated in different traffic conditions including various turning movements and volume splits between the two intersected streets. And at the same time, the features of queue length, control delay and service time will be investigated as well.

To fulfill the primary objective, this research focused on:

1. Collect data during afternoon peak hours at FWSC intersections in Lawrence and Overland Park using video cameras. Nine FWSC intersections with single lane approaches were chosen according to traffic conditions (volumes, continuous queues) and intersection geometries (single- lane approach, level grade and no parking).
2. A computer aided data processing program was developed using Visual Basic 6.0, based on what data would be needed in the simulation models. It extracted data from the Video files converted from the original videos taken at the six FWSC intersections.
3. After the related data were extracted from the video files using the data processing program, traffic flow characteristics and driver behaviors were analyzed using statistical techniques. Data analysis included the arrival patterns, move-up time, hesitation time under 27 traffic conflict situations, queue lengths, service time, control delays, deceleration and acceleration, and driving speed characteristics.

4. Based on the data analysis, simulation models were developed using Visual Basic. Individual vehicle turning movements and vehicle interactions were simulated. The simulation model represents the traffic flow patterns and driver behaviors, which can duplicate the statistic features of the collected data.
5. The simulation results were evaluated based on the collected data six intersections in Lawrence: queue length, service time, and control delay. Also, the simulation model was validated against the data collected at two intersections in Lawrence and one intersection in Overland Park. At the same time, two proposed queue models for AWSC intersections by Tian were evaluated. The 95th percentile queue lengths were calculated from the two models with the outputs from the simulation model and the HCM 2000 model. Then the results were compared with the field observations.
6. The traffic operation characters at FWSC intersections were analyzed using the simulation model.
7. Based on the 95th percentile queue lengths from the field observations, the simulation model was compared with several widely used software, including aaSIDRA, Synchro/SimTraffic and TSIS/CORSIM.
8. With the validated data, the capacity simulation program was developed to predict the capacities at FWSC intersections with single lane approaches. The capacities were predicted for a variety of traffic conditions, including various volume directional distributions, volume splits, and different turning movement compositions.
9. The simulation capacities were compared to the results from the HCM 2000 model (approach-based iteration model) and the addition-conflict-flow (ACF) model. At the same time, the capacities from Richardson and Hebert's predictions and the AWSIM

model were evaluated.

10. Several controversial points were addressed during this study:

    a) Do FWSC intersections have better performance under evenly distributed traffic on the two intersected streets?

    b) Does the HCM 2000 underestimate the capacity at FWSC intersections?

    c) Do right turn vehicles and left turn vehicles have an impact on the capacity at FWSC intersections?

    d) Which queue length model works better for FWSC intersections?

1.3 RESEARCH METHODOLOGY

This research encompassed a comprehensive study of traffic flow and operational patterns at FWSC intersections and combined traffic studies, analysis theories, and computer simulation, which required a knowledge of traffic engineering, applied mathematics, and computer science.

A turning movement combination based simulation approach was employed to estimate the capacities at FWSC intersections and related traffic studies were conducted. Figure 1.1 shows the flow chart of this research, which includes four phases.

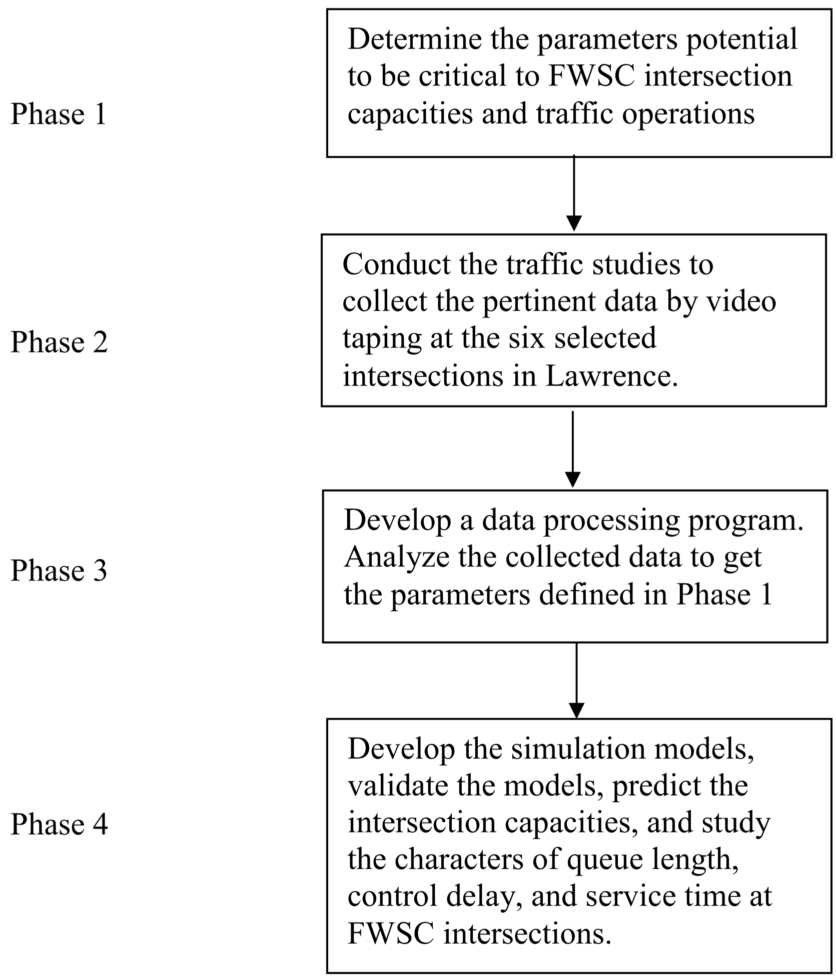


Figure 1.1 Research Methodology Flow Chart

In Phase 1, based on the scope of this study, the parameters to be used in simulation models were determined. According to the definitions in Phase 1, related traffic studies were conducted using video cameras at the six selected FWSC intersections in Lawrence. In Phase 2, based on the needed data defined in Phase 1, a data processing program was developed to extract data from video files and record the data into Excel spreadsheets. After the related data were collected, the parameter values or patterns were analyzed using statistical techniques in Phase 3. Phase 4 included developing the simulation models based on the collected data, including arrival pattern, and the hesitation time for the 27 turning movement combinations. Then the simulation models were tested against the field observations. At the same time, the simulated results were compared to the results from the 2000 HCM model, the ACF model and several other studies. In addition, the queue lengths from different models and software were evaluated.

1.4 DISSERTATION OUTLINE

The dissertation contains six chapters, including this introduction chapter. Chapter 2 includes a review of the capacity models, traffic patterns and parameters, simulation techniques and models. Also, their advantages and disadvantages are discussed. Chapter 3 determines the parameters to be used in the simulation model, including parameters from the traffic studies in this research and those from other research. It also covers the traffic study design for FWSC intersections. Chapter 4 presents the collected data and analysis results. The advantages of this research compared to other research are elaborated. Chapter 5 develops the simulation model to estimate the 95th percentile queue length, control delay and service time at FWSC intersections. The simulation results are compared with the observed data and the validation of the simulation model is conducted as well. At the same time, traffic operational characteristics at FWSC intersections were investigated using the simulation model and the HCM 2000 model. In addition, two queuing models were evaluated. The simulation queue lengths were compared with the results from other models or programs. Based on the validated data, a capacity simulation model was developed to predict capacities at FWSC intersections under a wide range of traffic conditions. Then, these results are compared with those from other theoretical or simulation models. In the end, Chapter 6 contains conclusions and summaries reached in this research. Also, some recommendations for future research are made in this chapter.

CHAPTER 2 LITERATURE REVIEW

2.1 PROBLEMS IN ESTIMATING FWSC INTERSECTION CAPACITY

FWSC intersections are very common in USA and widely used as a safe and traffic-calming device for neighborhood traffic. A FWSC intersection requires each driver to stop completely before preceding into the intersection. They are very important for both urban and suburban areas especially in mid-size and small cities. In the Manual on Uniform Traffic Control Device (MUTCD) there are specific criteria for FWSC installations. One of the applications is the lower threshold of the traffic volumes during certain periods. However, the capacity of the FWSC intersection is still problematic.

The techniques used in FWSC intersection capacity estimation have been improved compared to the original empirical study by Hebert in 1963. However, this development is relatively slower than that for signalized and two-way- stop sign controlled (TWSC) intersections. Partly, it is because that there is no clear right-of-way assignment except first-in-first-out and yield- to- the- right rules. Meanwhile, traffic operations at FWSC intersections are more complex in the real world because of driver behavior. Under capacity operation, it was commonly observed that the two intersected streets alternatively shift right-of-way. The two vehicles on the two opposite approaches can share the intersection depending on their turning movements. Different turning movement combinations of the two vehicles affect the hesitation time for the second vehicle to proceed into the intersection. Similar phenomena can be observed when the two vehicles are right or left conflicted. Therefore, turning movement combinations are important for studying FWSC intersections.

Generally, the factors that affect the FWSC intersection capacity include driver behaviors under Four-Way-Stop control, geometric conditions and traffic characteristics. A good model should incorporate all of these factors. Nevertheless, because of their uncertainty and complexity, it is almost impossible for a model to include all the factors.

Historically, three techniques, namely an empirical technique, an analysis technique and a simulation technique, have been used in FWSC intersection capacity estimations.

2.2 EMPIRICAL MODELS

When the theoretic analysis is limited, an empirical technique is usually a good choice. The development of empirical model depends on the collection of field data, mathematical statistics, and professional analysis.

Jacques Hebert's model [1963] was always cited as the first capacity model for FWSC intersections used in the U.S.A and his model became the basis for the capacity guidelines in the 1985 HCM [TRB, 1985]. In 1963, he collected departure headway data at three right-angle intersections in the Chicago metropolitan area for 80 minutes each using a movie camera. Through the data analysis, he concluded that left-turning vehicles had no effect on the capacity. For single lane FWSC intersections, he measured departure headways at two volume splits, 50/50 and 60/40. The headways were 7.65 sec and 7.15 sec respectively. According to the traffic conditions faced by the driver on the subject approach, he studied the headways for three cases, L headways (two intersected streets were loaded), N headways (only the subject approach was loaded) and I headways (subject approach was loaded and interfered by crossing street vehicles). Based on the headway data and mathematical analysis, a capacity model was presented to predict the capacity for single-lane FWSC intersections, which clarified that the volume split was the most influential factor and the only one incorporated in the model. Equation (2.1) shows the capacity equation for the total capacity of single-lane FWSC intersections.

$$C = \frac{7200}{(10.15 - 5S)S} \tag{2.1}$$

Where, C is the total intersection capacity; S is the ratio of the traffic volume on the major street to the total intersection volume. Table 2.1 lists the capacity of the intersections at different volume splits.

Table 2.1 Single-Lane FWSC Intersection Capacity Based on Various Volume Splits

Volume split	Capacity (vph)
50/50	1900
55/45	1800
60/40	1700
65/35	1600

As the first capacity model for FWSC intersections, Hebert model had both advantages and disadvantages. First of all, he suggested a capacity equation based on collected field data, which was a valuable reference for later research. In addition, he found that volume splits had a significant impact on the intersection capacity. But on the other hand, his conclusion that left-turning vehicles had no effect on capacity has been questionable. In all three cases, headways did not include the consideration of the turning movements and heavy vehicles. Therefore, the estimated capacities are not sensitive to the turning movements. His last shortcoming is the use of two intersections to establish a mathematical equation, which is very limited. It is very difficult to represent a wide range of conditions based on such a small sample size.

Between 1987 and 1989, the University of Idaho conducted a saturation headway study at

FWSC intersections and developed a capacity equation based on regression analysis [Kyte et al, 1989, 1990, and 1994]. Their conclusions and equations were published in Transportation Research Circular (TRC) 373[1991], which was the basis of the 1994 HCM update. The 1994 HCM expanded the traffic conditions faced by the subject approach drivers from two cases (one in which the driver faces another vehicle on the opposing or conflicting approaches and the other in which there are no vehicles on the other approaches) to four cases. In Case 1, vehicles are only on the subject approach. Case 2 occurs when there are vehicles only on the subject approach and the opposing approach. Case 3 occurs when the subject vehicle faces only conflicting vehicles. In Case 4, there are vehicles on all approaches. Figure 2.1 gives the definition of the intersection approaches and Table 2.2 shows the four cases and their corresponding saturated headways.

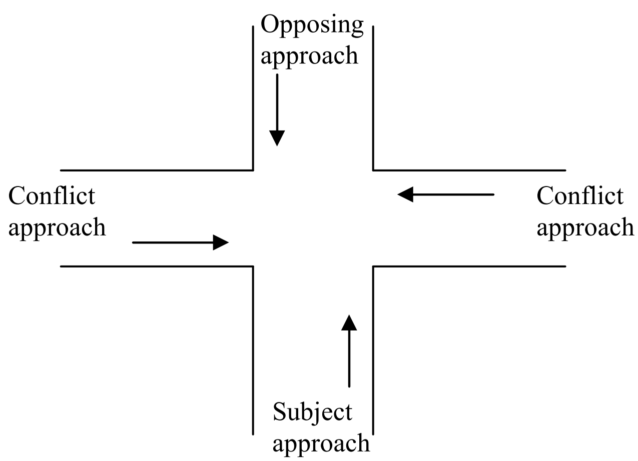


Figure 2.1 Definitions of Intersection Approaches

Meanwhile, the 1994 HCM update indicated that the departure headway was a function of the conditions present on the other intersection approaches, which implied an interaction between different approach vehicles. Table 2.2 shows the saturation headways for different cases. The capacity and delay equations are given in Equation (2.2) and (2.3)[1994 HCM].

Table 2.2 Variation of Saturation Headway with Number of Approach Lanes

Condition	Mean saturation headway (sec/veh)			
	Case 1	Case 2	Case 3	Case 4
All data	3.5	5.5	6.5	9
Single lane	3.9	5.6	6.5	9
Multilane	1.5	4.3	6.3	9.3

$$C = 1000V_{ps} + 700V_{po} + 200L_s - 100L_o - 300LT_{po} + 200RT_{po} - 300LT_{pc} + 300RT_{pc} \quad (2.2)$$

$$D = e^{3.8/v/c} \quad (2.3)$$

Where, C is the capacity of the subject approach (vph); D is the average total delay on the subject approach (sec/veh); L_s is the number of lanes on subject approach; L_o is the number of lanes; V is the volume on the subject approach (veh/hr); LT_{po} is the proportion of volume on the opposing approach turning left; LT_{pc} is the proportion of volume on the conflicting approaches turning left; RT_{po} is the proportion of volume on the opposing approach turning right; RT_{pc} is the proportion of volume on the conflicting approaches turning right; V_{ps} is the proportion of intersection volume on the subject approach, and V_{po} is the proportion of intersection volume on the opposing approach.

This approach is more complicated and advanced than Hebert's model in considering more factors and cases. But it did not directly consider the turning traffic interactions and the impact of the heavy vehicles, although it provided adjustment factors for turning movements.

2.3 ANALYTICAL MODELS

Anthony Richardson [1987] developed an analytical delay model by improving M/G/1 (Poisson or Exponentially distributed interarrival times, general process times, and a single server) queuing model using Pollaczek-khinthine formula based on the data collected by Hebert. Through iterations, the capacity can be estimated when the average service times reach equilibrium. Capacities calculated by his model are pretty close to Hebert's estimation.

Although Richardson's model is valuable for later analytical model development, it has similar limitations as Hebert's model in that the model did not consider heavy vehicle impacts, turning movement interactions, and other traffic cases due to the limited data.

Zion M. et al (1989) evaluated Richardson's delay model against the collected field approach delay data. Because the service time in Richardson's model is the moving-up time from the 2^{nd} position to the first position and the stopped delay is less than the approach delay, the study indicated that the delay model fit the field data and provided a lower bound of all the observations. Also, it concluded that the average delay should decrease as volume splits approach a balanced 50/50 split.

In the NCHRP 3-46 report, eight traffic cases based on the traffic conditions faced by the subject driver were considered [Kyte, M. et al, 1997,1999]. Later, they were combined into five cases according to the headway data, which were used in the 1997 HCM update [1997 HCM]. Table 2.3 shows the five cases. Using Richardson's probability principle, an iterative capacity model was developed to compute the capacities for each approach. At first, the degree of saturation for each approach was calculated using Equation (2.4) with a given initial situation headway value. Then, headway was computed using Equation (2.5) until the headway reached convergence.

$$x = Vh_d/3600 \tag{2.4}$$

$$h_d = \sum_{i=1}^{5} P[C_i]h_{si} \tag{2.5}$$

Where, x is the degree of saturation; V is the flow rate; h_d is the departure headway; $P[C_i]$ is the probability of degree of conflict for Case i; and h_{si} is the saturation headway for Case i.

Table 2.3 Conflict Cases Used in 1997 HCM Update and 2000 HCM

No. of conflict case	Vehicles on approaches			
	Subject	Opposing	Conflicting-Left	Conflicting-Right
1	Y	N	N	N
2	Y	Y	N	N
3	Y	N	Y	N
4	Y	Y	N	N
4	Y	Y	Y	N
4	Y	N	Y	Y
5	Y	Y	Y	Y

The 1997 HCM update and the 2000 HCM [2000 HCM] adopted Kyte's capacity model and Akcelik's delay model [2002]. The methodology is an improvement with larger data samples, more traffic cases, adjustments for heavy vehicles, and clear parameter definitions. However, it considers only the traffic conditions on approaches and neglects the turning vehicle interactions. Therefore, it cannot estimate the degree of conflict objectively. Also, it is not sensitive to the turning movements and the street splits [Wu, N, 2002]. Another shortcoming of the 2000 HCM is that it does not provide how to calculate queue lengths for FWSC intersections.

Moreover, for single-lane FWSC intersections, approaches with or without exclusive left turn lanes are considered equally. Wu, N. [2000] indicated that the left turn lane could significantly affect the intersection capacity when the volume split was not even.

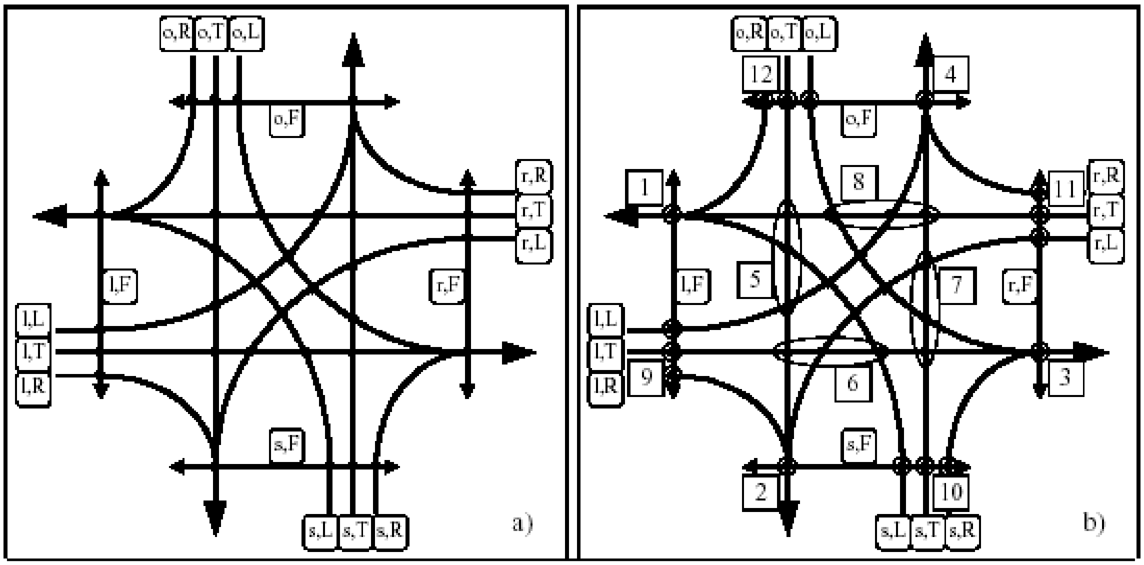


Figure 2.2 Intersections with 12 Vehicle and 4 Pedestrian Streams and the Critical Conflict Areas Between the Streams (From [Wu, N. 2000])

In Germany, another different technique has been employed in capacity calculations for FWSC intersections. This approach is based on the Addition-Conflict-Flow (ACF) theory, whose mathematic background is graph theory [Wu, N., 2000]. Also, Wu used the ACF technique in other kinds of unsignalized intersection analyses [2001, 1999]. For AWSC intersections, the conflict areas and conflict streams are analyzed in Figure 2.2 for single-lane approach AWSC intersections. According to the occupied time by a vehicle in the conflict areas and conflict streams, the stream capacity can be calculated using Equation (2.6), (2.7), and (2.8)(assuming there are no pedestrians). Then the approach capacity is computed based on shared lane capacity [Wu, N., 1999] using Equation (2.9). The capacities estimated from the ACF theory are given in Table 2.4.

$$C_{s.L} = \max\begin{cases} \dfrac{3600}{t_B} - \max\left[(Q_{o,R} + Q_{r,T}), (Q_{o,T} + Q_{r,T} + Q_{l,L}), (Q_{o,T} + Q_{r,L} + Q_{l,T})\right] \\ \dfrac{3600}{4 \cdot t_B} \end{cases} \quad (2.6)$$

$$C_{s.T} = \max\begin{cases} \dfrac{3600}{t_B} - \max\left[(Q_{r,R} + Q_{l,L}), (Q_{o,L} + Q_{r,L} + Q_{l,T}), (Q_{o,L} + Q_{r,T} + Q_{l,L})\right] \\ \dfrac{3600}{4 \cdot t_B} \end{cases} \quad (2.7)$$

$$C_{s.R} = \max\begin{cases} \dfrac{3600}{t_B} - \max\left[(Q_{o,L} + Q_{l,T})\right] \\ \dfrac{3600}{3 \cdot t_B} \end{cases} \quad (2.8)$$

$$C_s = \frac{Q_{s,L} + Q_{s,T} + Q_{s,R}}{X_{s,L} + X_{s,T} + X_{s,R}} \quad (2.9)$$

Where, C is the capacity for subject approach streams; Q is the flow rate for a certain stream; X is the degree of saturation of a certain stream; and t_B is the average departure headway.

Table 2.4 Capacities for Single-Lane AWSC Intersections Based on Wu's ACF Model

Turning composition Volume split	50/50	70/30	100/0
0% L-100%T-0%R	1960	1960	1960
20%L-60%T-20%R	1881	1699	1470

The ACF approach is easier to understand and use than the HCM method. Another merit of this method is that it can be applied to more complicate lane and traffic conditions. Its results for single-lane FWSC intersections agree with other research findings, to some extent. However, with the ACF approach, the capacity is independent of the volume splits when there are only through vehicles on the streets. This conclusion is not consistent with Kyte's simulation results

[2001].

Based on the ACF theory, Wu, N. [2002] re-categorized the 192 stream-based conflict cases into five conflict cases, which were the same as those used in the 2000 HCM. Iterative steps were used to achieve the stable saturated headways based on their probabilities of occurrence and the capacities were estimated. The comparison between the results from this modified model and the HCM model is shown in Figure 2.3.

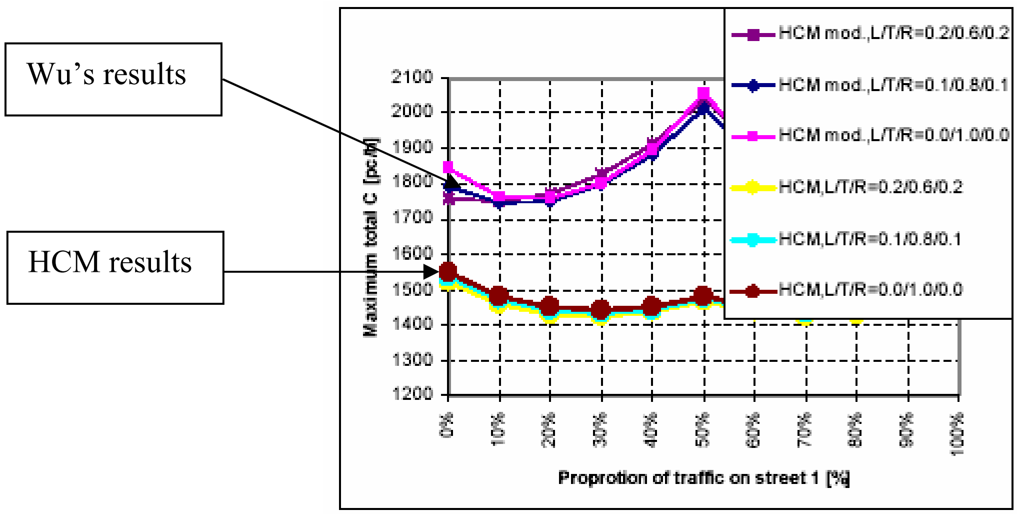


Figure 2.3 Total Capacity of the Intersection: Comparison Between HCM Model and the Modified HCM Model (from [Wu, N., 2002])

The modified HCM model has significant higher capacities than the HCM model because it considers different turning movements. Unlike the ACF model, the estimated capacities are sensitive to the volume splits when there are only through vehicles on both streets. The HCM model seems to underestimate the total capacities of FWSC intersections, which may be contributed by overestimating some conflict conditions. Therefore, the modified HCM model seems to provide more reliable capacity estimations than the HCM model. However, there are no data supporting the categorization from 192 cases to 5 cases and it may underestimate the conflicts. For instance, it considers that there is no conflict between the subject left turn vehicles and the opposing left turn vehicles, which contradicts what was found in observations at single-lane FWSC intersections [Kyte, M., et al., 1996].

2.4 SIMULATION MODELS

A simulation is the imitation of the operation of a real-world process or system over time [Banks, J. et al, 2000]. Traffic simulation techniques have been used since the early days of the development of traffic theory [Akcelik, M., 1997, Pursula, M., 1999]. They are powerful tools for the application of complex and dynamic models of travel and activity behavior due to the system complexity and uncertainty involved in the transportation system [Axhausen, K. et al, 1997].

"Simulation models predict system performance on the basis of a representation of the temporal or spatial interactions between system components, or both, often capturing the stochastic nature of traffic" [Lily, E., et al, 2000]. Several simulation models have been used in FWSC intersection capacity estimations. Bristow [1974] used computer simulation to determine the capacity at FWSC intersections. He considered three kinds of "cheats" or turning vehicles, which could share with or interrupt other vehicles when proceeding into the intersection. But his conclusion that the intersections with 50/50 volume splits have lower capacity than those with uneven volume splits seems to contradict the theoretical analysis.

Savur, V.S. et al (1977) developed the Traffic Experimental and Analysis Simulation (TEXAS) model to determine the capacities at unsignalized intersections [Thomas W. et al, 1977 and Lee, C. et al, 1979]. The simulation results [Salter, R. J. et al, 1991] indicate that the average delay for vehicles is the lowest when the volume split is 100/0 and highest when the volume split is even, which is opposite to the AWSIM' simulation results [Kyte, M., et al, 2001]. The TEXAS model is a very elaborate simulation model and a general tool for analysis of signalized and unsignalized intersections. Kyte's informal study showed that the delays from the TEXAS model were inconsistent with those measured in the field [Kyte, .M. et al, 1996]. It was indicated in Kyte's study that the TEXAS model significantly overestimate vehicle delay with high traffic volumes greater than 280 vph, but, on the other hand, it generally underestimates vehicle delay with low traffic volumes less than 100 vph.

Chan, Y. et al [1989] developed the STOP-4 simulation program based on field-collected data. Interestingly, their simulation results are similar to Bristow's findings that the capacity of the intersection increases when volume splits get uneven. It was found that it always predicts higher delay than the field observations [Zion, M., et al, 1990]

Another FWSC intersection simulation software, AWSIM, was formulated in 1996 [Kyte, M., et al, 1996]. This software is based on a large data sample collected for NCHRP Project 3-46. It assumes that no conflicts occurred in the conflict area or conflict points. Based on this assumption, the vehicles can enter the intersection separately or simultaneously. The capacities estimated from this model confirm that the intersection capacity is the highest when volume splits are even. Using this simulation model, the effects of the arrival distributions on the delay were studied. Also, the empirical delay and queue length models were developed based on simulation [Kyte, M., et al, 2001]. Although its capacity was close to the 1997 HCM, it only considered that the right turn movements could share with other vehicles. Other conditions under which the vehicles can share the intersection were not included. Another shortcoming worth mentioning is that the stopped delay was used in the simulation instead of control delay, which was not consistent with the 2000 HCM.

John, M. et al [1997] used General Purpose Simulation Software (GPSS) and field data to design a simple simulation program. There were no capacity data presented in the paper.

However, it pointed out that the HCM model underestimated the delays because it omitted the interactions of vehicles.

As discussed above, the empirical and analytical techniques are not sufficient to describe the traffic operations at FWSC intersections because of the complexities of driver behaviors. However, simulation may provide a better approach to estimate capacities at FWSC intersections. Because the existing simulation models have their limitations in predicting FWSC intersection capacities, queue lengths, control delays and service time, more reliable simulation models for FWSC intersection were developed to estimate intersection capacities and study traffic operational characters in this research. Prior to the simulation model development, the related traffic characteristics and parameters need to be determined by data collection and analysis.

CHAPTER 3 PARAMETERS USED IN SIMULATION MODEL

3.1 DATA COLLECTED IN THE FIELD

There are many factors affecting the traffic operations at a FWSC intersection, including location factors, geometric factors, operation factors, and traffic composition factors [Savur, V.S., 1977]. The location factors deal with the city population, access control, the distance from the other intersections and area type. The geometric factors are related to the number of legs, lane widths, grades, and having separate left or right turn lanes. The operation factors include one-way or two-way streets, parking on the approaches, speed limits, and peak hour factors. The traffic composition depends on volumes, heavy vehicles, turning traffics, pedestrians and arrival types. It is very difficult for a model to incorporate all these factors. In this research, the scope is limited to single lane (with width no less than 12 ft) approach FWSC intersections within midsize cities like Lawrence, Kansas. Parking is not permitted on the intersection approaches and there are no (or few) pedestrians crossing at the intersections.

Based on this scope, the parameters to be collected for each approach are: total volumes, turning volumes, the percentage of heavy vehicle, approach speed, peak hour factor, approach headway distribution, control delay, service time, and queue length. Additional data to be collected at intersection include: move-up time, hesitation time for different turning movement combinations, deceleration distance, acceleration distance, deceleration and acceleration time.

3.2 DEFINITION AND METHODS

This research does not involve steep grade intersections; with the maximum grades at the selected intersections about 3%. The volumes, headways, heavy vehicle percentages and approaching speeds can be collected directly from the data processing program.

3.2.1 Peak Hour Factor

Because the capacity is estimated based on hourly volume, the peak hour factor (PHF) must be provided to convert the average volume to demand flow rates. The peak hour factor is the ratio between the peak 15-minute flow rate and the full hourly volume as shown in the following equations:

$$PHF = \frac{\text{Hourly volume}}{\text{Peak flow rate within the hour}} \tag{3.1}$$

For 15-minute periods are used, the PHF is defined as:

$$PHF = V/(4 \times V_{15}) \tag{3.2}$$

Where,

V = peak-hour volume (vph);

V_{15} = volume during the peak 15 minutes of flow (veh/15 minutes).

The 2000 HCM demonstrates three approaches that can be used in a given study, including single analysis period T=15 min, single analysis period T=60 min and multiple analysis periods t=15 min. In this study, the single analysis period, T=15 min, approach is used to develop and validate the simulation model, in which a PHF values from the collected data and peak hour flow rates are used.

3.2.2 Saturated Departure Headway and Service Time

Saturated departure headway is the time between two successive vehicles departing from the same stop line with a continuous queue present. It is the function of the turning type, vehicle type, driver behavior, and the traffic conditions on the other approaches. It includes the move-up time from the second place to the first place and service time. The service time and move-up time are read from the video. According to the relationship between them (see Figure 3.1), the service time can be calculated from equation (3.3) (HCM 2000):

$$t_s = h_d - t_{mv} \tag{3.3}$$

Where, t_s, h_d, and t_{mv} are the service time, departure headway and move-up time, respectively.

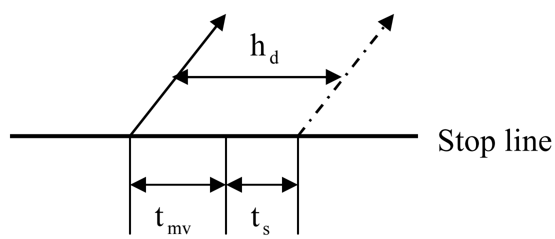


Figure 3.1 Relationships Between Departure Headway, Move-up Time, and Service Time

Based on the 192 combinations from the graphic theory by Wu, N. (2002), saturated departure headways can be theoretically recorded for every case. But it is impossible to get the data as combinations of turning movements at a real FWSC intersection because of the complication of the reality. Based on the fact that the hesitation time for an arrival car is related to possible conflicting vehicles on other approaches at the intersection, the data can be collected on the basis of the combinations of these two cars' turning movements. In this research, 27 combinations were used and details will be described in the next Chapter.

3.2.3 The 95th Percentile Queue Length

The 95th percentile queue is defined as the queue length that has only a 5 percent probability of being exceeded during a certain time period. It is a very important parameter to evaluate the performance of an intersection. The time interval of 20 sec. was used to collect the queue length samples for one-hour durations. Then the data were sorted in ascending order and the 95th percentile queue length was obtained based on the sample size.

3.2.4 Queue Delay, Stopped Delay, and Control Delay

Queue delay is the time difference between the time a vehicle enters the queue and it arrives at the stop line. In other words, it can be defined as the time delay from when a vehicle stops or joins the back of the queue to the instant that the vehicle crosses the intersection stop line (Yunlong Zhang, et al, 2001).

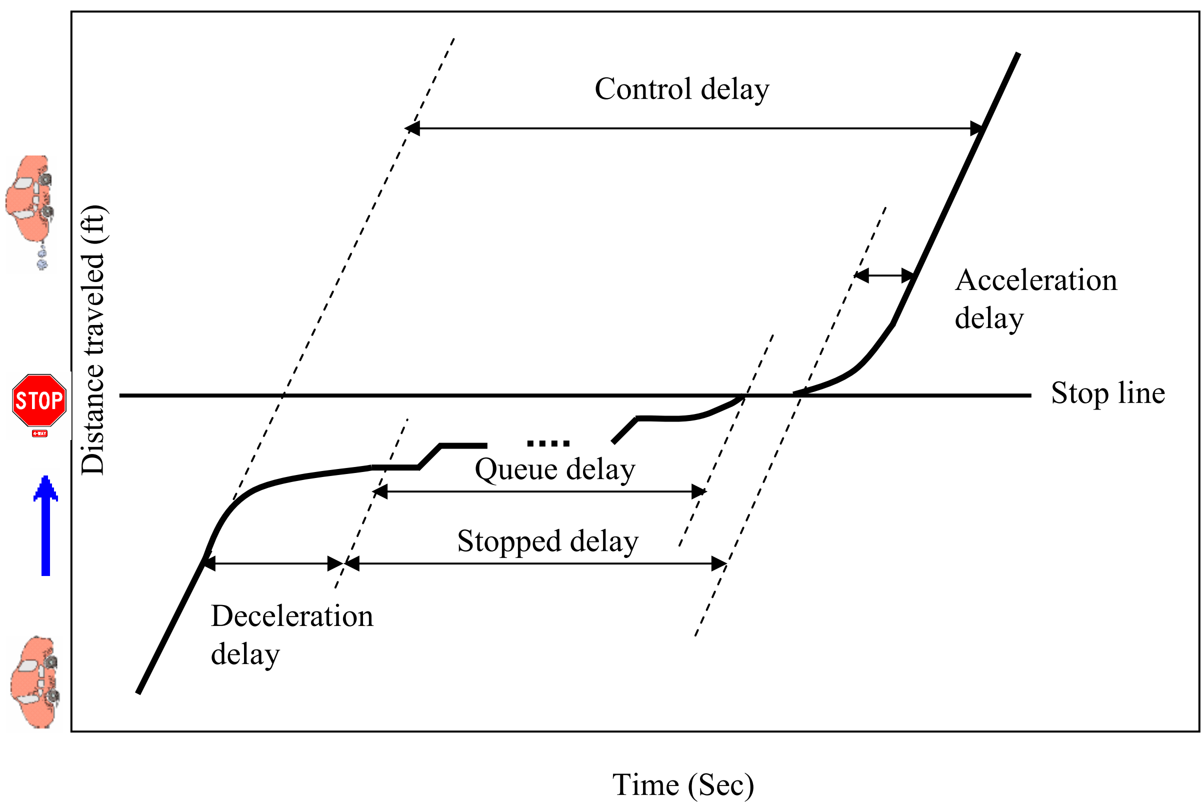


Figure 3.2 Delay Definitions

According to the 2000 HCM, stopped delay is a portion of control delay when vehicles are at a complete stop. For a signalized intersection, stop delay only considers the time lost when a vehicle stops in the queue waiting for a green signal or waiting for its leader to move forward (Yun long Zhang, et al, 2000). However, for FWSC intersections, the vehicles in the queue move-up one position and stop to wait for the next movement and so on. Therefore, it is difficult to measure the real stopped time. In the TEXAS model and the CORSIM model, a stop is identified when the speed of the vehicle is below 2 mph or 3 ft/sec. It is common that the speed of a vehicle moving forward in the queue in front of a stop sign is very slow. In practice, a vehicle that crawls forward in a queue or that has a gap of less than three vehicle length with respect to its leader vehicle may also be treated as being stopped (Quiroga, C, 1999). Thus, the time difference between when a vehicle stops at the stop sign or joins the queue to when the vehicle departures from the stop line can be defined as stopped delay. The AWSIM defines this delay as total delay, which is confusing because the total delay usually refers to control delay (Mousa, M, 2002). In this model, the stopped delay is the sum of the queue delay and service time. This is different from the CORSIM model, in which queue delay is always greater than stopped delay.

The 2000 HCM defines control dealy as a portion of the total delay and it includes deceleration delay, queue delay, move-up time, stopped delay and acceleration delay. Total delay is the time difference actually experienced by the vehicle and the reference travel time in the absence of traffic control, geometric delay, any incidents , and any other vehicles. It was found that only a few studies dealt with control delay definitions and measurements. These studies included the 2000 HCM and the work carried out by Yunlong Zhang(2000, 2002), and Ragab, M. (2002, 2003). Control delay is ofen referred to as total delay, which is illustrated in Figure 3.2.

3.2.5 Control Delay Collection Method

Based on the control delay definition, it includes deceleration delay and stopped delay as well as acceleration delay, which can be calculated by the difference between the travel time without delay and the actual travel time over the distance from the upstream unaffected point to the downstream unaffected point. The actual travel time can be measured using a test vehicle. The question is how to determine the unaffected point. Quiroga (1999) used the global positioning system (GPS) to measure the test car delays and determined unaffected distances for signalized intersections. The 2000 HCM provides a method based on the number of the vehicles in the queue in fixed intervals to get control delay data at signalized intersection, which can get only rational delay results. Another method used before was tracing vehicle trajectories, which was laborious and time consuming (Olszewski, P., 1993). Akcelik (2002) derived a polynomial model of acceleration and deceleration profiles for signalized intersections. The model required many parameters and a lot of data collection. Because of the difficulties in collecting acceleration and deceleration data, uniform acceleration and deceleration were often used.

Ragab (2002) presented a method to measure stopped delay as well as acceleration and deceleration delay at signalized intersections. One person with an audiocassette recorder was assigned at each of the 12 screen lines (see figure 3.3). The vehicle was randomly selected and traced from the first to the 12th screen line.

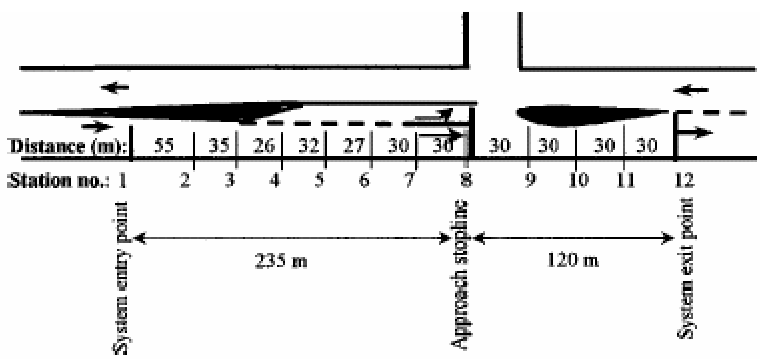


Figure 3.3 Screen Lines Used for Data Collection (from Ragab, 2002)

This approach is very laborious, although it allows for calculating the speed and accelerate rate at any point between the 12 screen lines.

In order to simplify the process, this study used a laser speed gun, which can measure the speed and distance at the same time. The speeds and distances to the stop line were read and recorded in a video camera. D1 and D2 in Figure 3.4 are the deceleration distance and acceleration distance, respectively. They were approximately determined first by the readings of the laser speed gun. It was found that 500 feet is sufficient to cover the acceleration distance or the deceleration distance for intersections that do not have long queues. After processing all the reading data, D1 and D2 were established to cover the whole acceleration and deceleration distance. A distance of 400 feet for both D1 and D2 was used in this study, which will be discussed in the next Chapter.

Using the data processing program, the time when the target vehicle's speed and distance were read can be derived. The time when a vehicle stops at the stop line or joins the back of the queue was recorded as well as the time when the vehicle departures from the stop line. In the mean time, distances and speeds were manually put in the same excel file where the time data were recorded. Then the acceleration rate and deceleration rate were calculated based on the speeds and the time. In addition, the control delay was calculated according to Figure 3.2.

3.2.6 Equipment and Design Set Up

This research used an Automatic Video-Recording system installed on one trailer. The video camera is located on the top of the mast, which can reach about 25ft high and can be adjusted to different heights based on the view requirement. Through cables the video images are recorded into tapes by the recorder in the cabinet. Because of its height, the camera can have a bird's view of the entire intersection. Therefore, one video camera was used to collect volume and turning movement data at one FWSC intersection during the PM peak hour. It also can cover the opposing approach. If there are long queues on other approaches, three other video cameras were used to cover the other three approaches. Figure 3.4 illustrates the schematic placement of the videos.

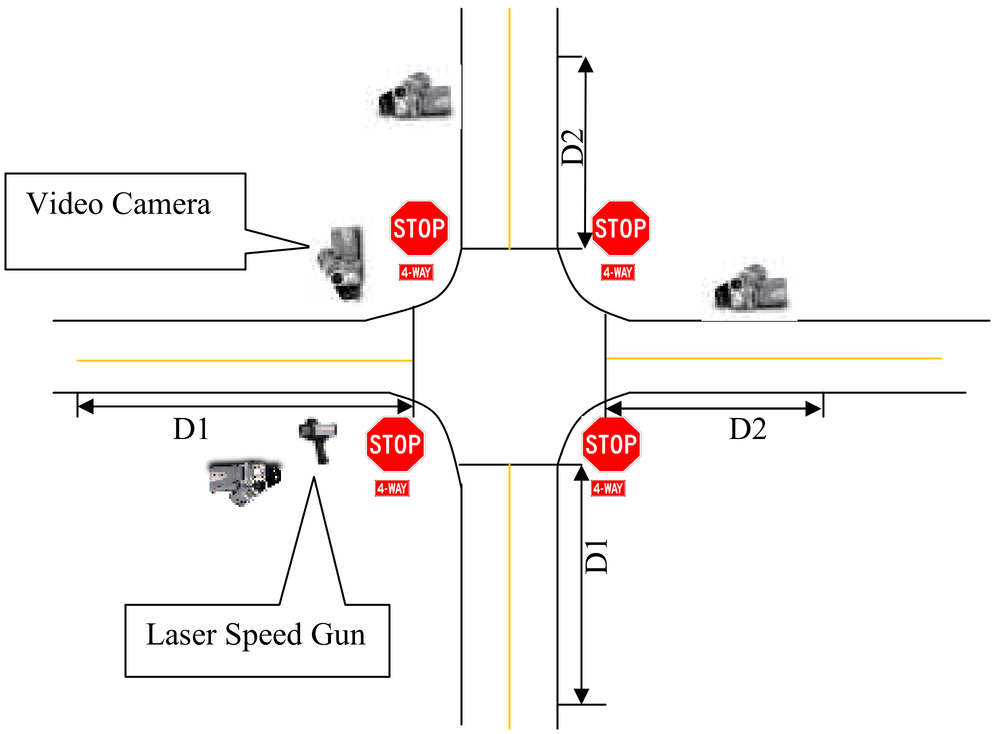


Figure 3.4 Video Cameras Set Up Design

3.2.7 Arrival Headway and Speed Distributions

After D1 was determined, the headways and speeds were collected on an upstream spot with the distance from the stop line a little longer than D1. Previous studies identified that the headway distribution is negative or a shifted negative exponential (Savur, V.S., 1977, and Kyte, M. et al, 1996) and the speed distribution is normal (Joe, Lee, 2001). The collected headway data were tested against these distribution models using the Chi-Square (X^2) goodness-of –fit test.

The Chi-Square test process can be described as follows:

1. H_0: The random variable, X, follows a specified distribution with the parameters given by estimates;

2. Arrange n observations into a set of k class intervals ($k \le \frac{n}{5}$);

3. Calculate the test statistic by: $X_0^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E^i}$, where, O_i is the observed frequency in the ith class interval and E_i is the expected frequency in that class interval;

4. Check if $X_0^2 > X_{\alpha,k-s-1}$. If so, the hypothesis will be rejected, otherwise, the hypothesis will be accepted.

After all the data were collected, data analyses were conducted and the results are presented in the next chapter.

CHAPTER 4 DATA COLLECTION AND ANALYSIS

4.1 SITE SELECTION

There are several types of FWSC intersections in terms of geometric features, including one lane in one direction with or without exclusive right turn or left turn lanes, and two lanes in one direction with or without exclusive turning lanes. To keep the scope of this research manageable, this study was confined to FWSC intersections with one lane in each direction without exclusive turning lanes. No parking on the streets, no or few pedestrians, and minimal platoon impacts were also limits in this research.

Lawrence is a college town, whose population is about 81,604. There are more than 50 AWSC intersections in Lawrence, but many of them allow parking on one or two of the intersected streets, some of them are 3-way stop sign controlled intersections. Based on the research scope, six FWSC intersections were selected. Among of the six intersections, the intersections of 15^{th} St. & Haskell, 15th St. & Barker Ave., 11^{th} St. & Connecticut St. are under capacity or near capacity operation at least on one approach. Figure 4.1 shows the selected intersection locations on the map. Figures 4.2 thru 4.7 show pictures of all the selected sites.

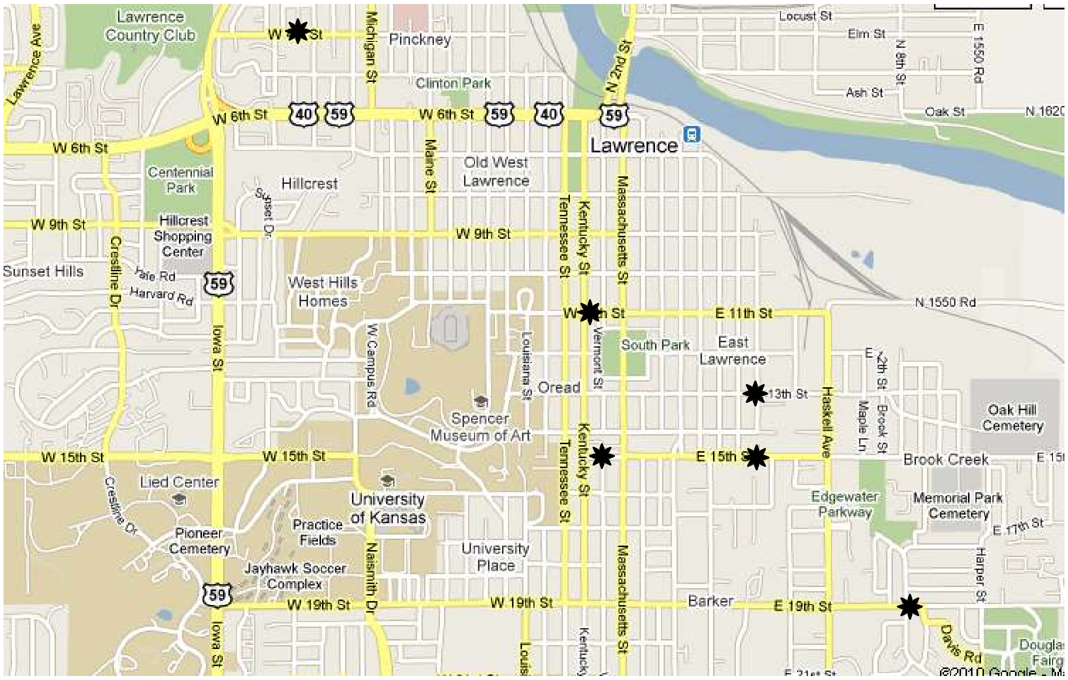


Figure 4.1 Selected Site Locations

Figure 4.2 Site 1: 15th Street and Haskell Avenue

Figure 4.3 Site 2: 19th Street and Harper Street

Figure 4.4 Site 3: 4th Street and Michigan Street

Figure 4.5 Site 4: 13th Street and Haskell Avenue

Figure 4.6 Site 5: 15th Street and Barker Avenue

Figure 4.7 Site 6: 11th Street and Connecticut Street

4.2 DATA PROCESSING PROGRAM

The data processing program developed for the research was used to extract data from videotapes taken at the chosen sites using digital Sony video cameras. Two procedures were involved. First, it was necessary to export the captured video to a computer in a WMV video file with Movie Maker 2 under a Windows XP environment. Then, the Visual Basic 6.0 program was used to manipulate the video file when it played back in Windows Media Player 6.0 or a higher version. The program used the features of Windows Media Player, including “play”, “pause” and “stop”. At the same time, the program displayed the total play-time of the video and the elapsed-time. It tracked the time when there was an event. For example, it recorded the time when the “stop” button was clicked and also the time when the “hesitation” button was clicked. This made it possible for the time differential to be recorded in an Excel spreadsheet and to calculated for needed parameters. In this program, all the required data were recorded in Excel spreadsheets, which allow data calculations and statistical analysis. Figure 4.8 indicates the interface of the data processing program.

Figure 4.8 The Data Processing Program Interface

The data processing program had several functions as following:

1. Count volumes, including the total volumes of the intersection and on every approach, and turning movements in 15-minute intervals were recoeded. Heavy vehicles were counted separately. If there is an existing queue, the vehicles in the queue were included in the first 15 min volumes.
2. The different turning movement combinations on the subject approach and the other three conflicted approaches and the time when the vehicles departed the stop line and entered the intersection were recorded. Then, the hesitation time for the 27 turning movement was analyzed based on the time differences.

3. The vehicles' passing time based on the time difference when the vehicle departs the stop line on subject approach and crosses the second stop line before it leaves the intersection was calculated.
4. The move-up time was determined by pressing the stop key "," when a vehicle in the second position in the queue started to move and pressing the start key "." when the vehicle stopped at the stop line. Move-up time is the time difference between the two events. This function also can easily track the time for a vehicle's movement in the queue.
5. Service time was calculated by the time difference when a vehicle stopped at the stop line and when it proceeded into the intersection. The stop key and start key were used to serve this purpose.
6. The time, the speed, and the distance were recorded when the control delays were investigated using the video file where the laser speed gun readings were recorded. In the meanwhile, the deceleration and acceleration were calculated.
7. Speeds also were derived from the video based on the travel time and the travel distance, 50 feet, which was marked by colorful tape and cones on the roadside.
8. Arrival headways were recorded by the time differences when the consecutive vehicles pass the same location.

4.3 DATA ANALYSIS

The data were collected during the 2004 spring. All the six intersections were video taped during the PM peak hours (4:00-6:00Pm) and in fine weather. Each intersection has four approaches and they are named as Approach 1, Approach 2, Approach 3, and Approach 4 in the data processing program. The same approach naming will be used in the simulation programs in the next Chapter. The approach definitions for each intersection are shown in Table 4.1.

Table 4.1 Approach Definitions for the Six Selected Intersections

Intersection	Approach 1	Approach 2	Approach 3	Approach 4
15th St. at Haskell Ave.	Haskell Ave. SB	15th St. EB	Haskell Ave. NB	15th St. WB
15th St. at Barker Ave.	15th ST. WB	Barker Ave. SB	15th ST. EB	Barker Ave. NB
13th St at Haskell Ave.	Haskell Ave. NB	13th ST. WB	Haskell Ave. SB	13th ST. EB
11th St. at Connecticut Ave.	Connecticut Ave. SB	11th St. EB	Connecticut Ave. NB	11th St. WB
19th at Harper St.	19th ST. WB	Harper St. SB	19th ST. EB	Harper St. SB
4th St. at Michigan St.	Michigan ST. NB	4th ST. WB	Michigan ST. SB	4th ST. EB

Table 4.2 Traffic Volumes at Six Selected Intersections

Intersection	Approach 1		Approach 2		Approach 3		Approach 4	
	Volume	PHF	Volume	PHF	Volume	PHF	Volume	PHF
15th St. at Haskell Ave.	313	0.89	400	0.96	422	0.94	251	0.9
15th St. at Barker Ave.	528	0.99	440	0.98	248	0.9	64	0.89
13th St at Haskell Ave.	368	0.87	26	0.77	301	0.84	6	0.41
11th St. at Connecticut Ave.	458	0.92	142	0.86	382	0.89	172	0.85
19th St. at Harper St.	81	0.84	144	0.83	143	0.8	302	0.93
4th St. at Michigan St.	166	0.8	143	0.73	225	0.95	86	0.86

4.3.1 Traffic Volumes

The volume data were collected at 15-minute intervals for an hour. The number of heavy vehicles was recorded at the same time. Then the adjustment factor of 1.7 (From NCHRP 3-46) was used for heavy vehicles. Table 4.2 summarizes the volumes and peak hour factors for each approach at the six selected intersections.

4.3.2 The 95th Percentile Queue Length and Service Time

The 95th percentile queue length and service time data were collected at the six selected intersections based on their definitions. Tables 4.3 and 4.4 tabulate the 95th percentile queue lengths and service time values for each approach.

Table 4.3 The 95th Percentile Queue Lengths at Six Selected Intersections

Intersection	Approach 1	Approach 2	Approach 3	Approach 4
15th St. at Haskell Ave.	6	8	14	4
15th St. at Barker Ave.	5	7	2	1
13th St at Haskell Ave.	3	2	1	0
11th St. at Connecticut Ave.	6	2	6	1
19th St. at Harper St.	1	1	1	1
4th St. at Michigan St.	1	1	1	1

Table 4.4 Service Time at Six Selected Intersections

Intersection	Approach 1	Approach 2	Approach 3	Approach 4
15th St. at Haskell Ave.	4.55	5.6	5.48	6.1
15th St. at Barker Ave.	3.13	3.86	4.4	2.83
13th St at Haskell Ave.	2.98	2.87	4.66	3.34
11th St. at Connecticut Ave.	2.01	1.84	2.29	2.82
19th St. at Harper St.	3.36	4.03	3.22	3.54
4th St. at Michigan St.	2.03	2.47	2.76	3.02

4.3.3 Move-up Time

Move-up time is defined as the time a vehicle moves from the second position to the first position. From the data collected from the six intersections, the maximum move-up time was 6.79 seconds and the minimum value wais 1.59 seconds. The average move-up time was 3.0 seconds, which was greater than the move-up time of 1.8 seconds used in AWSIM and the move-up time of 2.0 seconds used in the HCM 2000.

4.3.4 Hesitation Time

Hesitation time is the time needed by a vehicle in the first position at the stop line to enter the intersection because of a prior vehicle's movement on other approaches. It depends on the subject vehicle's turning movement and the movement of the vehicles that arrived at the intersection prior to the subject vehicle. There are 27 possible combinations at an AWSC intersection. Figure 4.9 and Table 4.5 show the all combinations and annotations.

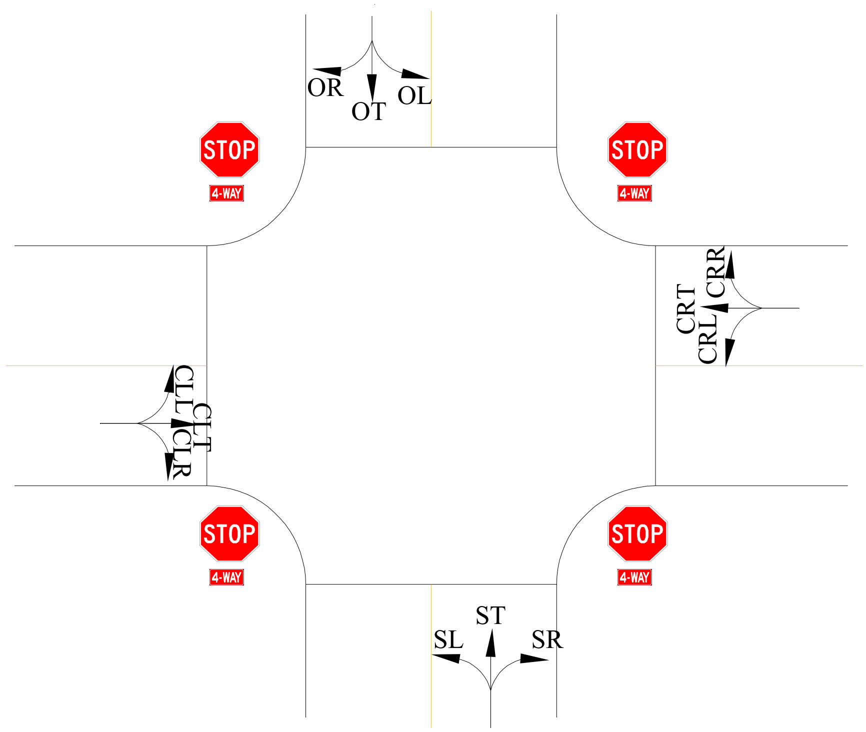


Figure 4.9 Different Turning Movement Annotations at ASWC

Where,

SR: right turn vehicles on the subject approach

ST: through vehicles on the subject approach

SL: left turn vehicles on the subject approach

CRR: right turn vehicles on the right conflict approach

CRT: through vehicles on the right conflict approach

CRL: left turn vehicles on the right conflict approach

CLR: right turn vehicles on the left conflict approach

CLT: through vehicles on the left conflict approach

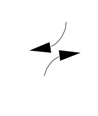
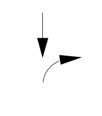
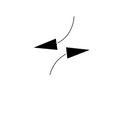
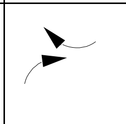
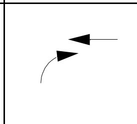
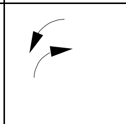
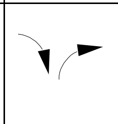
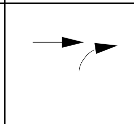
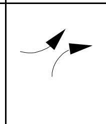
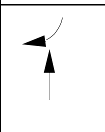
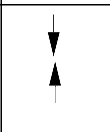
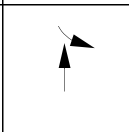
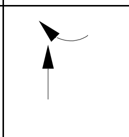
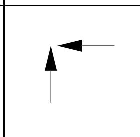
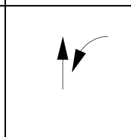
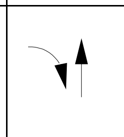
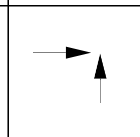
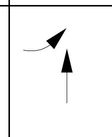
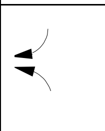
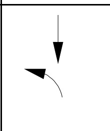
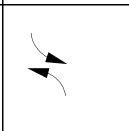
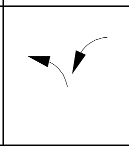
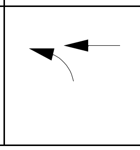
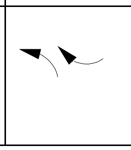
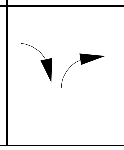
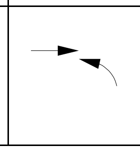
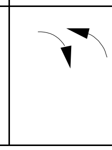
CLL: left turn vehicles on the left conflict approach

OR: right turn vehicles on the opposite approach

OT: through vehicles on the opposite approach

OL: left turn vehicles on the opposite approach

Table 4.5 27 Turning Movement Combinations at AWSC Intersections

	Opposite approach			Right conflict approach			Left conflict approach		
	OR	OT	OL	CRR	CRT	CRL	CLR	CLT	CLL
SR									
ST									
SL									

This definition is different from the hesitation time used in the AWSIM program, where the hesitation time was defined as the waiting time of the subject vehicle at the first position in front of the stop line according to the three traffic conditions on other approaches: no vehicle on the other approaches, vehicles on all the other approaches, and vehicles on more than one of the other approaches. The AWSM program did not categorize the hesitation time based on the turning movements of the subject vehicle and vehicles on the other approaches. On the contrast, Wu classified the combinations of the turning movements of the subject vehicle and the vehicles on the other approaches into 192 categories (ACF technique). It is not only trivial, but also few of these combinations can be accurately observed because the vehicle movements in the combinations can occur in different turns, which may lead to different results. In addition, these classifications need to be observed in true saturated traffic conditions. Otherwise, they are rarely observed. Bristow classified the traffic combinations into 27 categories. But only three types of cheats were used in his simulation, which could not describe the real traffic conditions.

There are several advantages in using the combinations of the subject vehicle and the vehicle entering the intersection prior to the subject vehicle. First of all, unlike the AWSIM program, there is no need to define the conflict area in the intersection as the AWSIM program did. Moreover, different turning movement combinations have different conflict areas. The hesitation time can show the difference between different turning movements. For example, there is one car on the opposing approach at an intersection when the subject vehicle arrives at the stop line and there are no vehicles on other approaches. It will take a longer time for the subject vehicle to proceed into the intersection if the first vehicle turns left than if it makes a right turn. Secondly, these combinations can indicate all different turning movements on the approaches to an AWSC intersection. Also, the turning combinations can be flexibly used for the vehicles on all four approaches by changing the subject approach instead of following the rigid 192 classifications in the Addition-Conflict-Flow technique (ACF). The data were extracted from the video when there was no pedestrian interference because of the research scope limitations. The observed data for these 27 turning movement combinations are shown in Table 4.6.

Table 4.6 Hesitation Time for the Subject Vehicle in Different Traffic Conditions

	Opposite approach			Right conflict approach			Left conflict approach		
	OR	OT	OL	CRR	CRT	CRL	CLR	CLT	CLL
SR	1.5	1.5	3.05	1.5	1.5	1.5	1.5	3.05	1.5
ST	1.5	2	2.52	2.8	2.5	3.8	1.5	3	3.25
SL	1.5	2.24	1.87	1.5	3.4	3.8	1.5	3.6	3.8

When a vehicle arrivals at the intersection, the driver first judges the traffic situation. If there are no conflict vehicles, the vehicle will proceed. The time value for the driver to make the judgment is 1.0 sec. when there is no vehicle on the other approaches. It is also noticed that there is no significant difference between different turning vehicles when there are no vehicles on the other approaches. When there is one or more vehicles on the other approaches, the driver will wait to enter the intersection after the conflict vehicles leave. The required judging time depends on the turning combinations of the vehicles listed in Table 4.5.

4.3.5 Speed Distribution

A normal distribution is the most common model used in speed distribution. It has the following probability density function:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp(-\frac{1}{2}(\frac{x-\mu}{\sigma})^2)$$

Where, μ is the mean of the collected data and σ^2 is the variance.

For the collected approach speed data on the Haskell Ave. site, the mean and variance were 26 mph and 16.32 mph, respectively. The distribution fitting and Chi-Square test results are shown in Figure 4.10 and they indicate a good fitting to a normal distribution with a computed chi-square value of 2.34 with a tabular value of 21.98 (α=0.05, f=8).

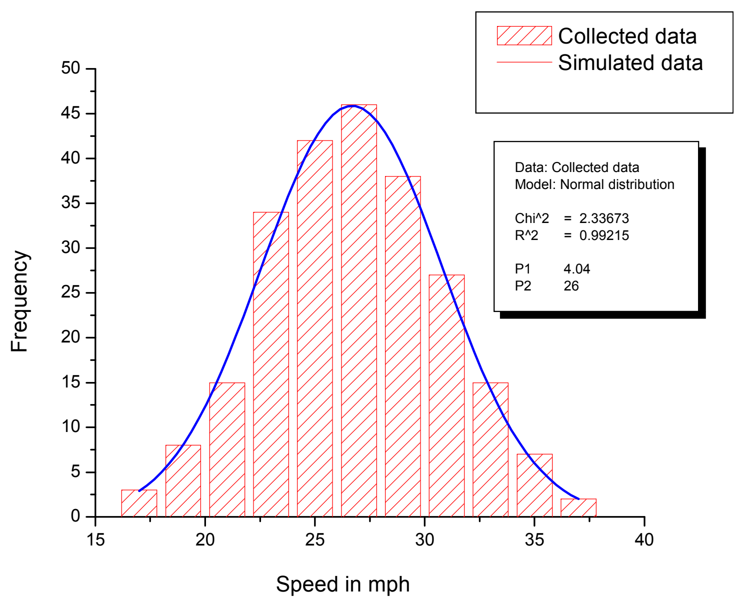


Figure 4.10 Speed Distribution Analysis

4.3.6 Headway Distribution

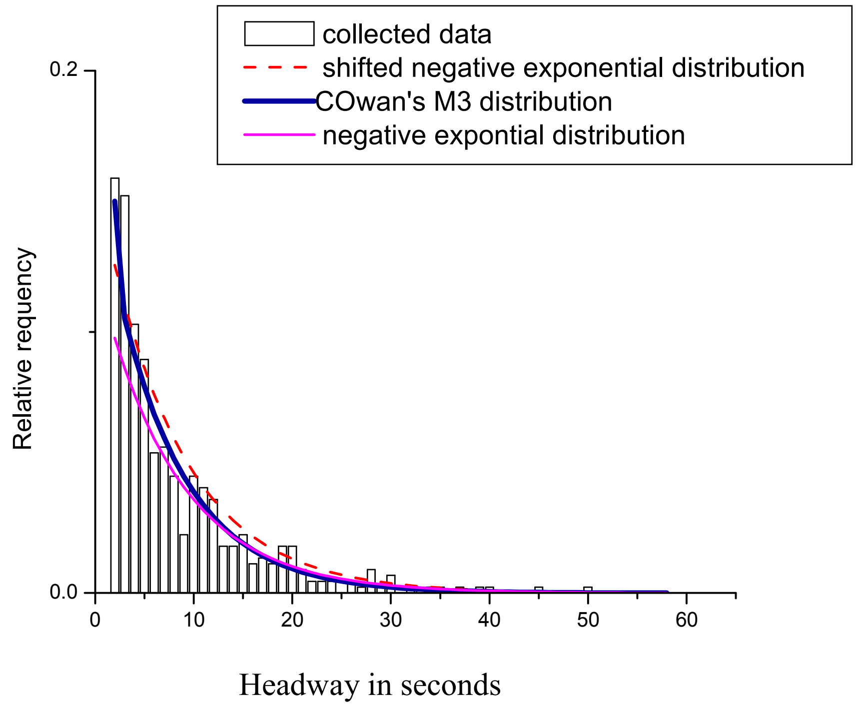


Figure 4.11 The Comparisons Between Collected Data and Three Models

Three different headway distributions, namely negative exponential, shifted negative exponential, and Cowan's M3 distribution, were used in previous studies. The collected headway data were tested against these three models using Microcal Origin 6.0. Figure 4.11 and Figure 4.12 show the histogram of the collected headway data at the intersection of 15th Street and Haskell Ave. and the comparisons between the collected data and the simulations of these three models. these figures indicated that the simulations of these three models are very close when the headways are large, but only Cowan's M3 distribution can fit the collected data well when the headway values are small.

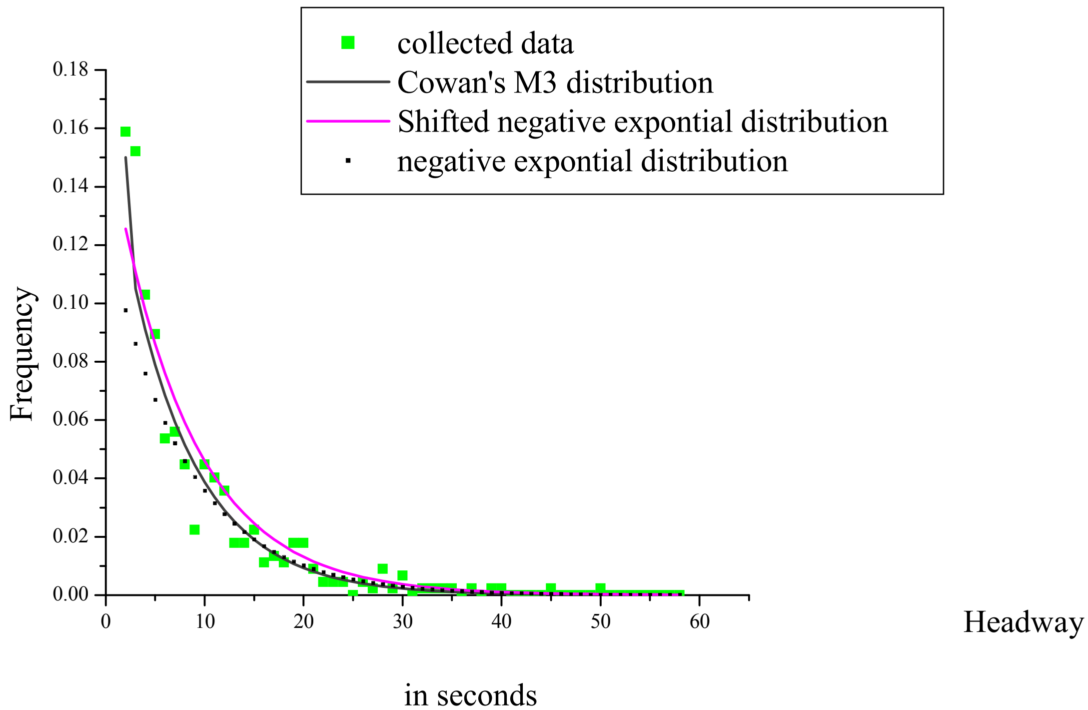


Figure 4.12 The Comparisons Between Collected Data and Three Models

The Cowan's M3 model was proposed by Cowan in 1975 and used extensively in traffic analysis. It describes a bunched exponential distribution of arrival headways. The cumulative distribution of Cowan's M3 model is given by:

$$F(t) = \begin{cases} 1-\varphi e^{-\lambda(t-\Delta)} & for \quad t \geq \Delta \\ 0 & for \quad t < \Delta \end{cases}$$

Where,

Δ = minimum arrival headway (seconds)

φ = proportion of free vehicles

λ = a decay parameter, $\lambda = \varphi q/(1-\Delta q)$, where, q is the arrival flow of the traffic lane,

The φ value can be estimated by generalizing the bunching implied by the negative exponential model as:

$$\varphi = e^{-b\Delta q}$$

Where, b is a bunching factor. Akcelik carried out a calibration and it was found that b is 0.6 for single lane case and Δ=1.5s. It was found that φ for the collected data is 0.89 and λ is 0.137. Also, the data were fitted in Microcal Origin and the fitted parameters are 0.85 and 0.135, respectively. It can be derived that b=0.86. With these two parameters, the collected data and the simulated data by Cowan's M3 modal are shown in Figure 4.13.

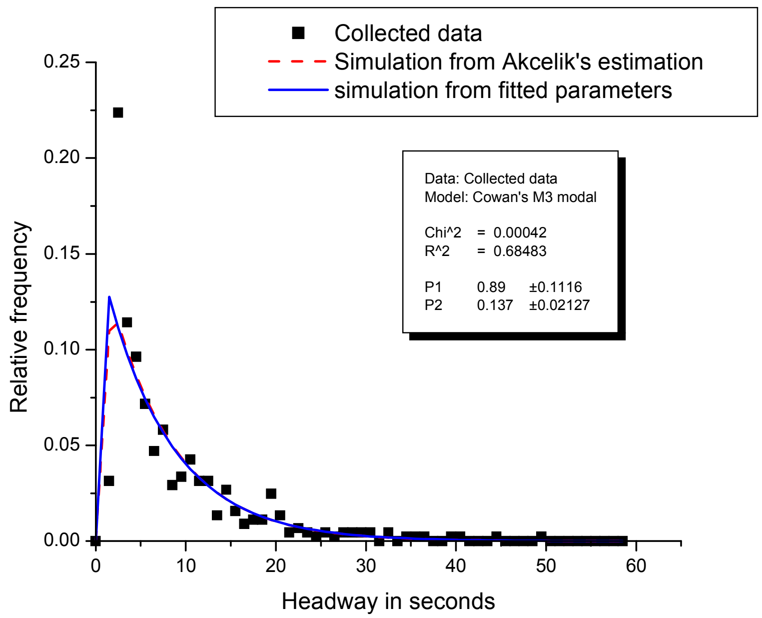


Figure 4.13 Simulated Headway Distributions

It can be seen that the two results from the fitting parameters and the parameters from Akcelik estimation are very close. However, it shows that the simulation data do not fit the collected data when the headways are small. Also, the statistical results from the Chi-square test and the regression test indicate the simulations do not fit very well.

If it is assumed that Δ = 2 s, then the φ and λ values are 0.86 and 0.144, respectively, from Akcelik's estimation, and 0.839 and 0.19, respectively, from Microcal Origin. It was found that b is 0.6. The simulation results are shown in Figure 4.14. It is shown that the simulation results fit the collected data very well.

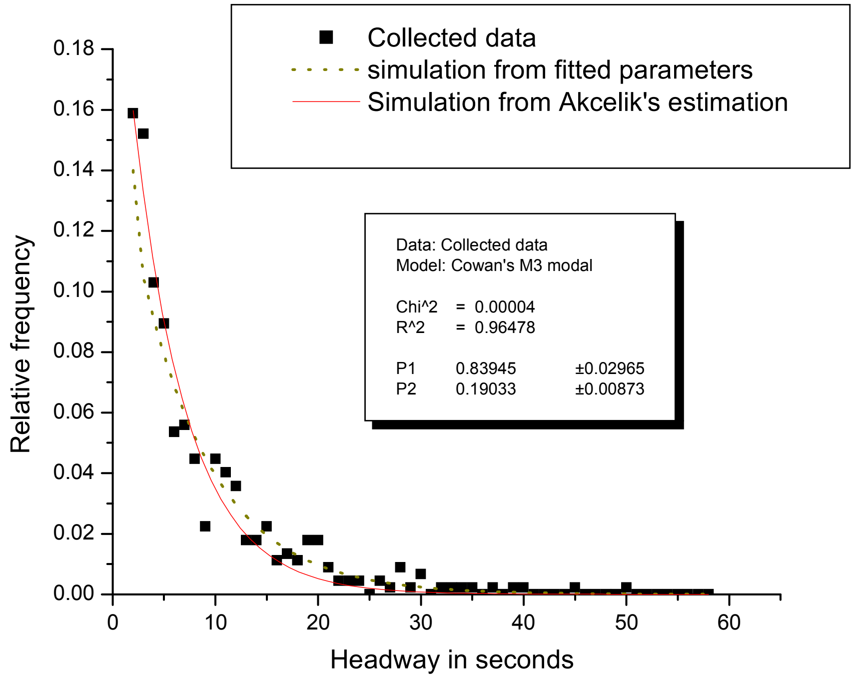


Figure 4.14 Simulated Headway Distributions

The probability density function of a Cowan's M3 model is:

$$f(t) = \varphi\lambda e^{-\lambda(t-\Delta)} \qquad \text{for } t > \Delta$$

$$= 1-\varphi \qquad \text{for } t=\Delta$$

$$= 0 \qquad \text{for } t< \Delta$$

By setting $\Delta = 0$ and $\varphi = 1$, the negative exponential distribution model can be derived and the shifted negative exponential distribution model can be derived when $\varphi = 1$.

From the data analyses above, the Cowan's M3 model will be used in the simulation and b = 0.6 will be used in deriving the λ and φ values.

4.3.7 Control Delay

The control delay data were collected at the intersection of 11th Street and Connecticut Street using three video cameras and a ProLaser II speed gun. The speed gun can cover up to one mile in ideal conditions with a beam width just 3.5 feet wide at 1000 feet. It can measure the vehicle speed between 2.5 mph and 300 mph. The speed and distance of a vehicle can be read from the speed gun. When the data were read, they were recorded on the video camera and the time can be extracted from the data processing program. Only one person is needed in the data collection and thus, it is convenient to decide the entry point and exit point because the speed gun can measure continuous speed and distance of a vehicle. Another reason for using this method is that the video camera can not cover such a long distance because of trees along the streets and the difficulty of locating a good parking spot for the surveillance trailer. A data sample was collected at the intersection of 15th Street and Haskell Ave. to decide the length that the control delay should cover. Figure 4.15 and Figure 4.16 show the relationship between the travel distance and the travel speed. As shown in Figure 4.15, when the distance between the approaching vehicles and the intersection stop line is 400 ft, the average speed was 29.6 mph, which is 98.7% of the speed limit of 30 mph. This distance is considered long enough for most vehicles to decelerate in the upstream of the intersection. As shown in Figure 4.16, when the distance between the exiting vehicles and the stop line is 400 ft, the average speed was 29.4 mph, which is 98% of the speed limit. It is also sufficient for most vehicles' acceleration in the down stream of the intersection.

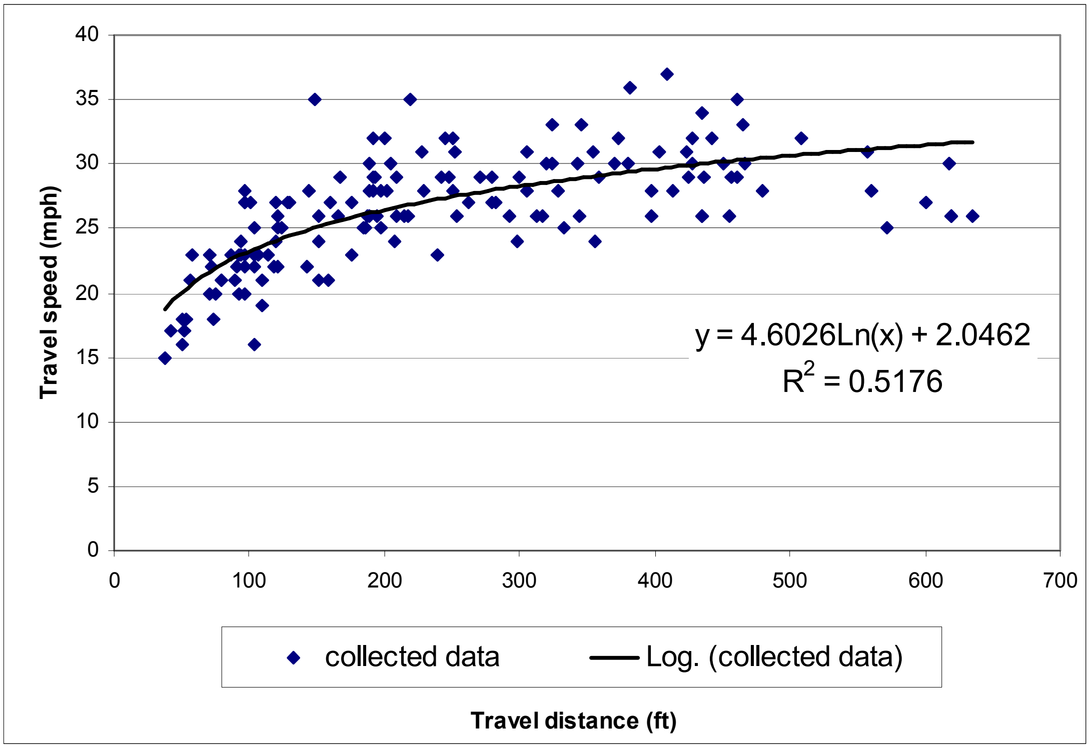


Figure 4.15 Speed Reduction with Travel Distance

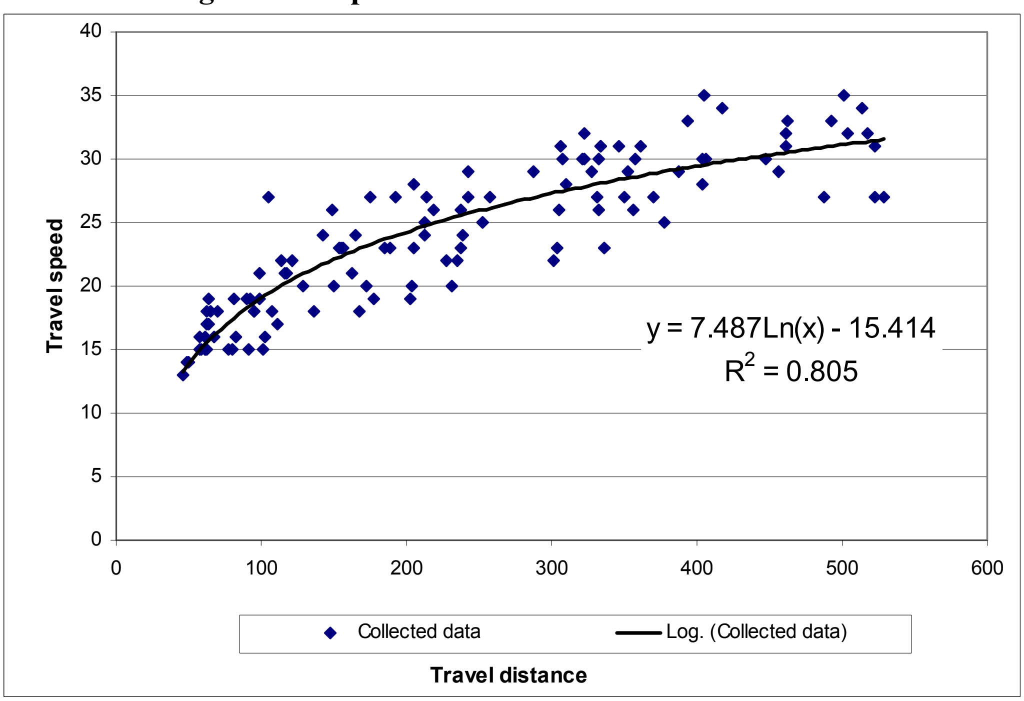


Figure 4.16 Speed Increase with Travel Distance

The control delay data were collected at the intersection of 11th Street and Connecticut

Street. Figure 4.17 shows the speed profile in the system. The average entry speed was 31.64 mph and the standard deviation was 3.80 mph. The average exiting speed was 29.14 mph, comprising about 92% of the average entry speed.

The relationship between travel time and travel speed is shown in Figure 4.18 and acceleration rates are showed in Figure 4.19. The average deceleration time was 10.34 sec., the maximum deceleration time was 13.41 sec., the minimum deceleration time was 8.12 sec. and the standard deviation was 1.34 sec. The average deceleration delay was 1.66 sec., the maximum deceleration delay was 4.6 sec. and the minimum deceleration delay was 0.31 sec., and the standard deviation was 0.91 sec. The average acceleration time was 13.44 sec., the maximum acceleration time was 17.66 sec. and the minimum acceleration time was 10.15 sec., and the standard deviation was 1.59 sec. The average acceleration delay was 4.08 sec., the minimum acceleration delay was 0.73 sec. and the maximum acceleration delay was 9.24 sec., and the standard deviation was 1.53 sec. The average deceleration rate ranged from 0.52 mph/s (0.77 ft/s^2) near the system entry point (400 ft from the intersection stop line) to 7.21 mph/s (10.57 ft/s^2) near the intersection stop line, with corresponding maximum deceleration rates of 2.7mph/s (3.99 ft/s^2) and 10.03 mph/s (14.72 ft/s^2), respectively. The average acceleration rate varied from 3.32mph (4.87 ft/s^2) at the start of acceleration to 0.8 mph/s (1.17 ft/s^2) with corresponding maximum acceleration rates of 6.55 mph/s (9.61 ft/s^2) and 1.84 mph/s and (2.71 ft/s^2). The average deceleration delay was less than acceleration delay and this might have been because of the higher rates of deceleration on approaches than acceleration rate through the intersection. The speeds, deceleration and acceleration values are presented in Table 4.7. The average deceleration-acceleration delay was 5.74 sec., which was comparable to the 5 sec. deceleration-acceleration delay used in the HCM 2000.

Table 4.7 Measured Acceleration and Deceleration Values

Values	Deceleration			Acceleration		
	Initial speed (mph)	Time (s)	Delay (s)	Final speed (mph)	Time (s)	Delay (s)
Average value	31.65	10.3	1.66	29.14	13.4	4.08
Standard deviation	3.8	1.34	0.9	3.59	1.59	1.53
Minimum value	25	8.12	0.31	24	10.2	0.72
Maximum value	41	13.4	4.61	39	17.7	9.24

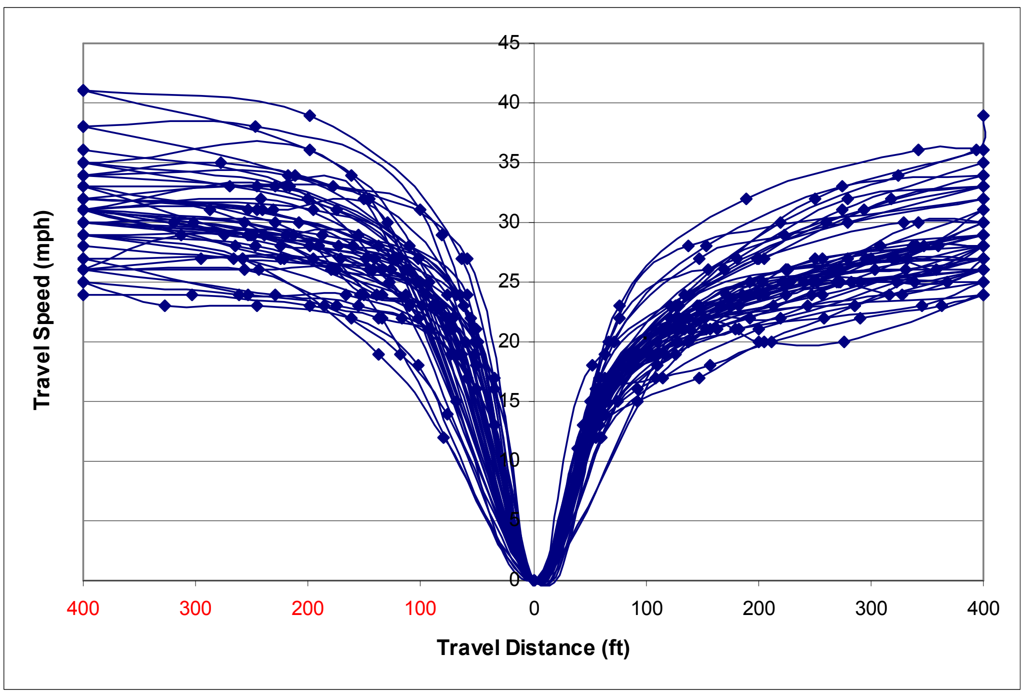


Figure 4.17 Speed Profile at FWSC Intersection

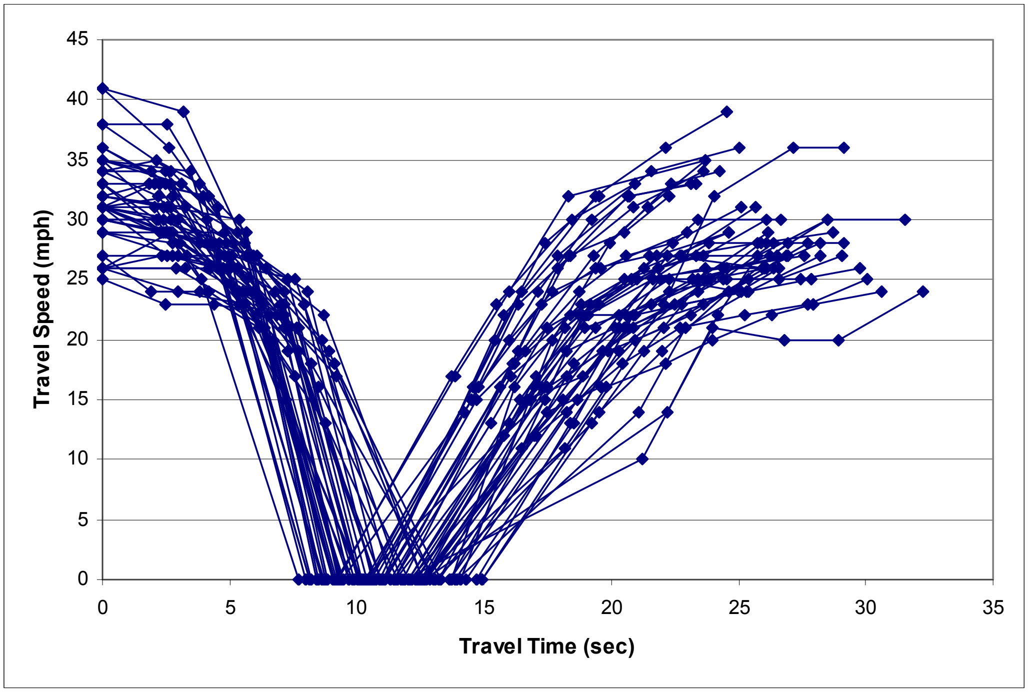


Figure 4.18 Travel Time and Travel Speed in System

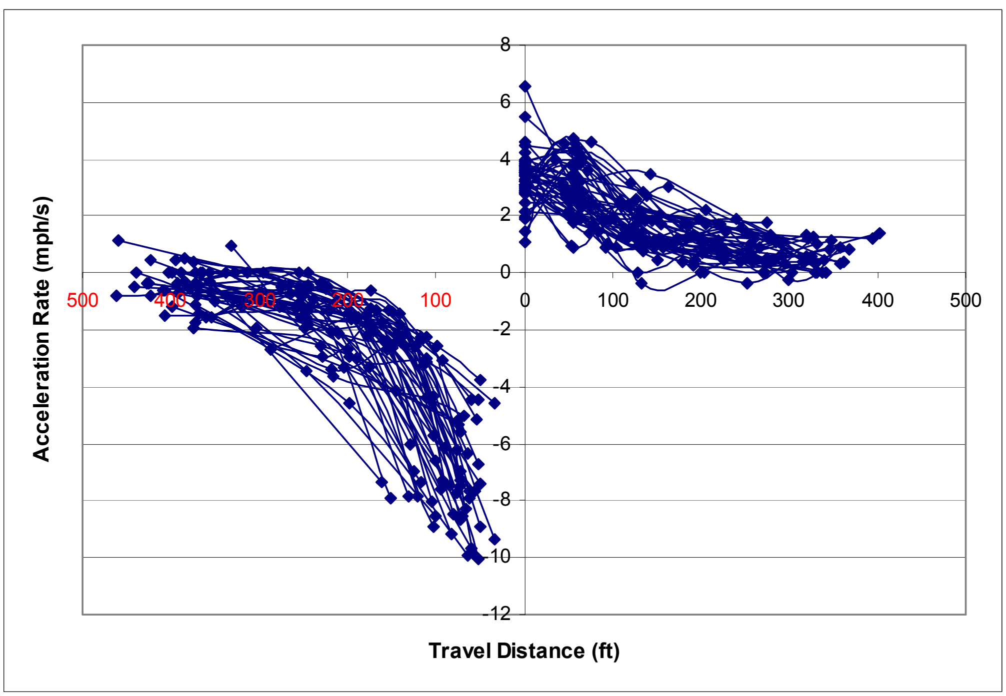


Figure 4.19 Acceleration Rate at FWSC Intersections

CHAPTER 5 SIMULATION MODEL AND RESULTS

5.1 INTROUDUCTION

Simulation is a dynamic presentation of some phenomena (systems) in the real world by a computer model through a period of time (Pursula, M., 1999). Since the early 1950's and 1960's, simulation has had a tremendous development in transportation research. CARSIM, TEXAS, and STOP-4 are some of the examples. Unlike analytical techniques that are normally limited by variable numbers, simulation can handle a wide variety of conditions and can capture interactions between drivers, vehicles, geometric conditions, and traffic control methods. The application of a micro-simulation offers a powerful tool for engineers to analyze the traffic operation of a network or a single spot location. Currently, simulations are widely used in transportation planning, transportation design and operations.

Several most popular and recent programs are TSIS, Synchro/SimTraffic, and VISSIM. TSIS contains the NETSIM and FRESIM programs and is mainly used for arterial streets and freeway traffic operation analysis. Synchro/SimTraffic is a window based program and widely used for signal timing design and evaluation. It incorporates the HCM 2000 and is widely used by traffic engineers. VISSIM was developed by PTV (Planung Transpor Verkehr) AG of Karlsruhe, Germany. It is a very powerful simulation program, which allows users to code most of the transportation scenarios. But for FWSC intersections, it is very difficult or impossible to simulate all the different 27 turning movements by defining the conflict areas without additional programming. The AWSIM program was developed for four way stop sign controlled intersections in 1996 by Kyte et al.. The program simulates the average queue length. It uses an approach concept and the conflict area at the center of an intersection as shown in Figure 5.1. It assumes that left turn vehicles have no effect on capacity. Moreover, the interactions between different turning movements are not considered. The simulated queue length data based on the AWSIM program are higher than the field data collected in NCHRP project 3-46.

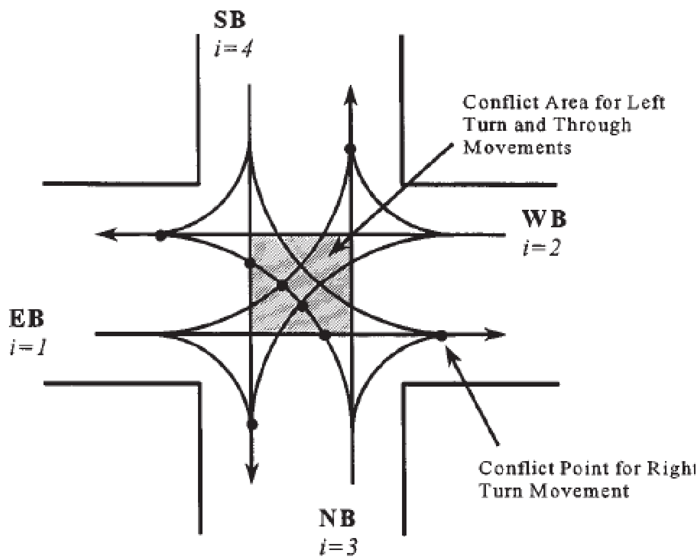


Figure 5.1 Traffic Conflict Area at AWSC Intersections (Kyte et al. 1996)

In order to incorporate the 27 turning movements in this research, VB 6.0 was used to program the simulation models. There are typically two approaches to generate a simulation model: a discrete event-based approach and a real time based approach. Compared to the real time based approach, the discrete event-based approach requires fewer steps and is much faster because less computer memory is required. Thus, the discrete event-based approach is frequently used in simulation models. But the events must be well defined for the traffic scenarios. In this research, the discrete event-based approach was used to "describe" the 27 turning movement combinations as described in Chapter 4. Queue and delay models and a capacity model were developed. Figure 5.2 presents the turning movements numbering system used in this study. From field observations and the HCM 2000, a two phase traffic operation, as shown in Figure 5.3, was used in the simulation model.

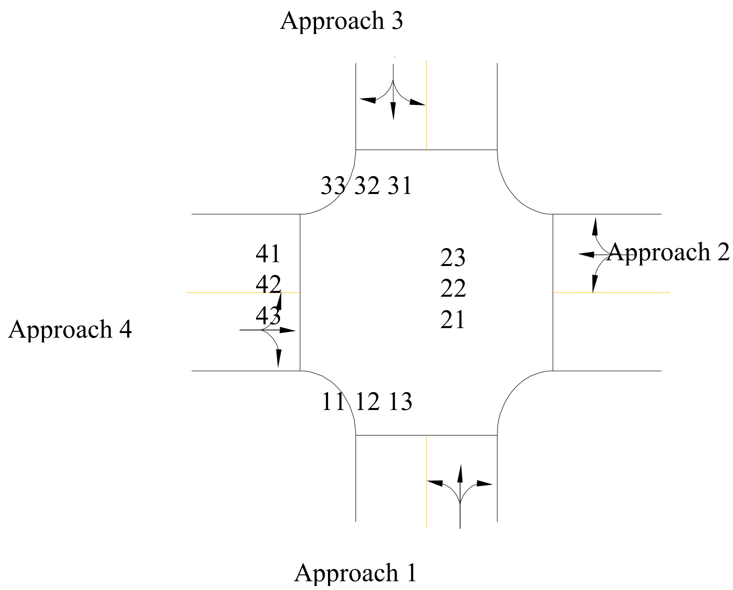


Figure 5.2 AWSC Intersection Turning Movements

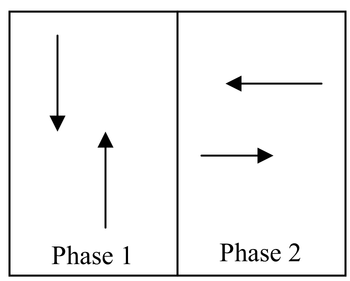


Figure 5.3 Two Phase Operation

5.2 QUEUE AND DELAY SIMULATION MODEL

Figure 5.4 illustrates the simulation procedures to obtain the 95th percentile queue length, the average control delay, and the average service time. A total of 30 minute simulation period was used so that simulation results can be compared with other research findings and the HCM (2000). The first 15-minute was used to develop an equilibrium condition. All the data were collected at the second 15-minute simulation period.

The Cowan's M3 headway distribution proved to be the best fit for the field data in Chapter 4. Based on the traffic demand, headways were calculated with the parameter b of 0.6 (if the distance between the intersection and a signalized intersection is less than 0.5 miles, b=1.5). "First-come, first-serve" and "Yield to the driver on your right" are the general operation rules used for AWSC intersections. The first vehicle is defined as the vehicle that arrived at the stop line first or the first vehicle in a queue. The second position is where the second vehicle in the queue is located. After the first vehicle proceeds into a intersection, the vehicle at the second position moves up to the stop line. Next, which vehicle proceeds into the intersection depends on traffic conditions on the other approaches, the turning movements, and the general rules as well.

Queue length is a very important parameter in evaluating existing intersections or comparing design alternatives for a new intersection. The 95^{th} percentile queue length is widely used as one of the design criteria. All the vehicles, which arrived earlier than the simulation clock, were in the queue. In this simulation, queue length was recorded whenever there was a change in queue length on one of the approaches. Then the 95^{th} percentile queue length was calculated based on the entire queue data collected during the simulation.

The control delay used in this simulation included four events: deceleration delay, queue delay, service time, and acceleration delay. The average deceleration delay and acceleration delay value of 5.74 seconds, as described in Chapter 4, was used in the simulation, while a value of 5-second was used in the HCM 2000 model. The queue delay included two parts: the time used by a vehicle to move to the second position from the end of the queue and the move-up time from the second position to the first position at the stop line. The 3–second move-up time was used based on the collected data in the field (Chapter 4). The queue delay value is derived from the time when a vehicle joins the back of the queue to the time when the vehicle departs the stop line and proceeds to the intersection. The service time is defined as the time difference between the time when a vehicle stops at the stop line and the time when the vehicle departs from the stop line. During simulation, three time parameters were recorded for each vehicle, the departure time t_d, the time t_e when the vehicle entered the queue system, and the time t_f when the vehicle got to the first position. The service time t_s and the queue delay t_q can be obtained as $t_d - t_f$ and $t_f - t_e$, respectively. Accordingly, the control delay t_c is equal to $t_q+t_s+5.74$ sec.

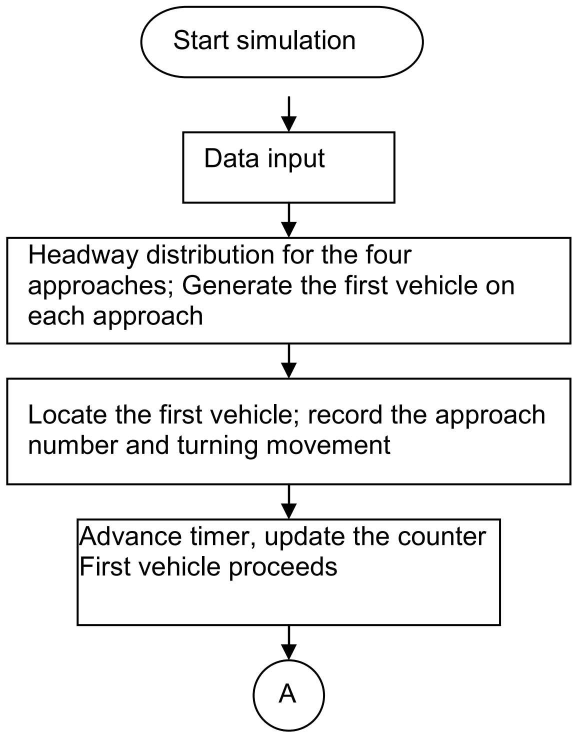


Figure 5.4 Simulation Procedures for Queue Length and Control Delay Model

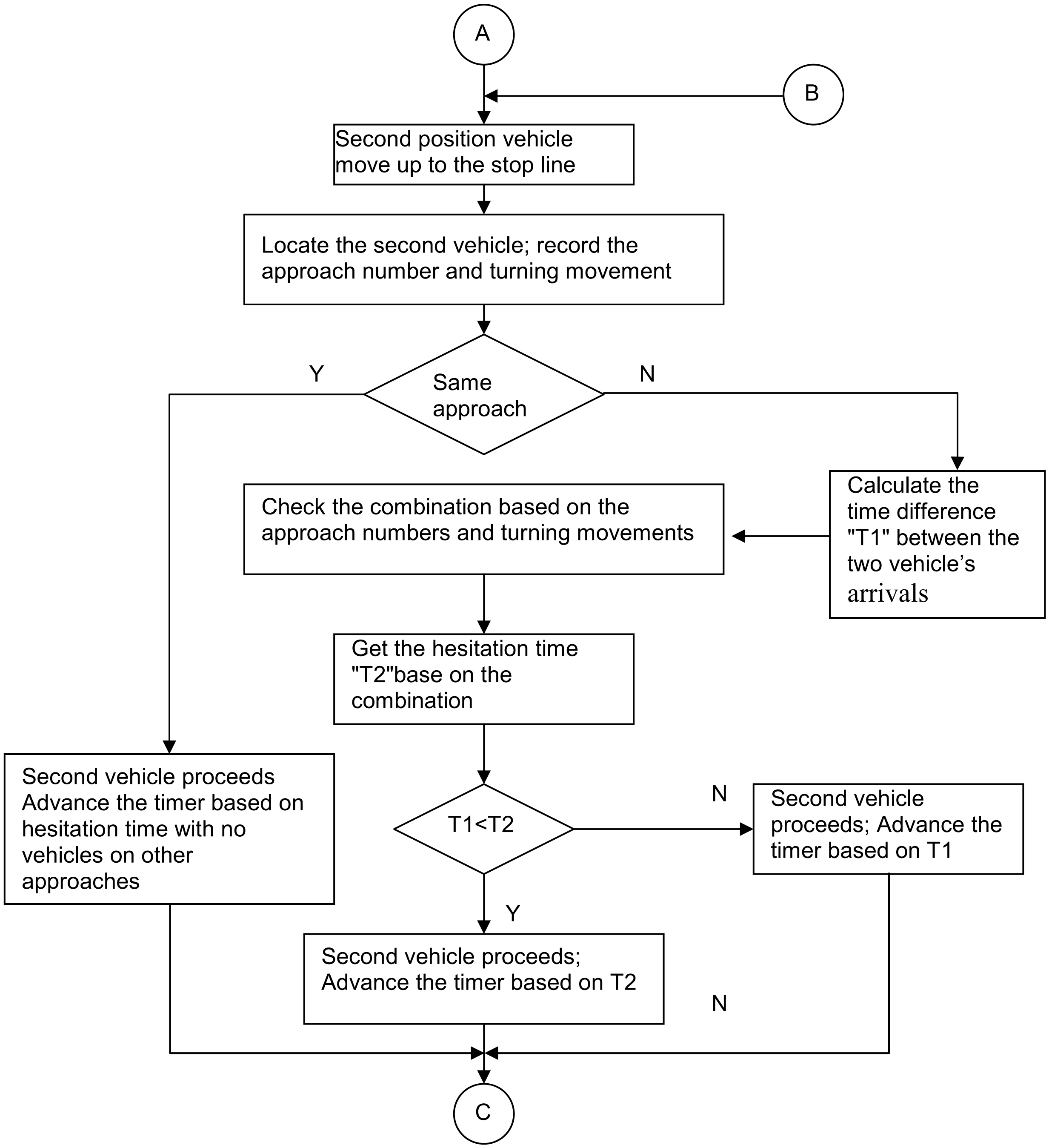


Figure 5.4 Simulation Procedures for Queue Length and Control Delay Model (Cont')

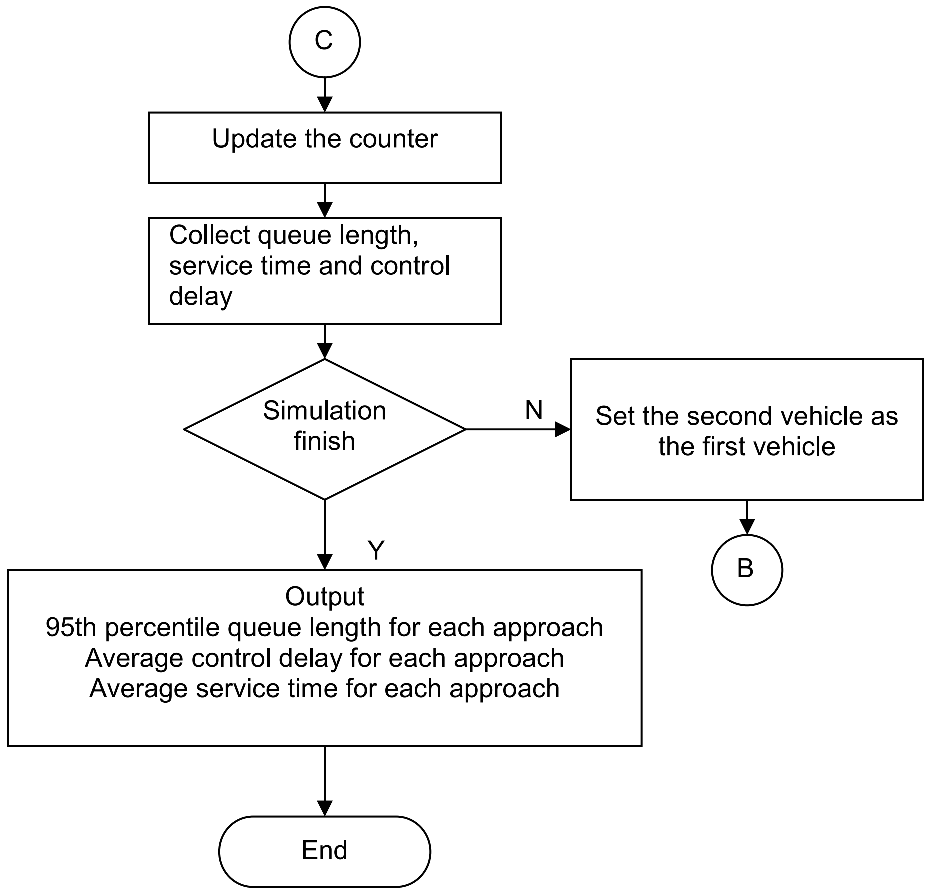


Figure 5.4 Simulation Procedures for Queue Length and Control Delay Model (Cont')

5.3 SIMULATION INPUTS AND OUTPUTS

Figure 5.5 shows the data input interface screen for the simulation model. The required input data were peak hour volumes on each approach based on peak hour factors (PHF) and turning volume percentages. For heavy vehicles, the adjustment factor of 1.7 (From NCHRP 3-46) was used.

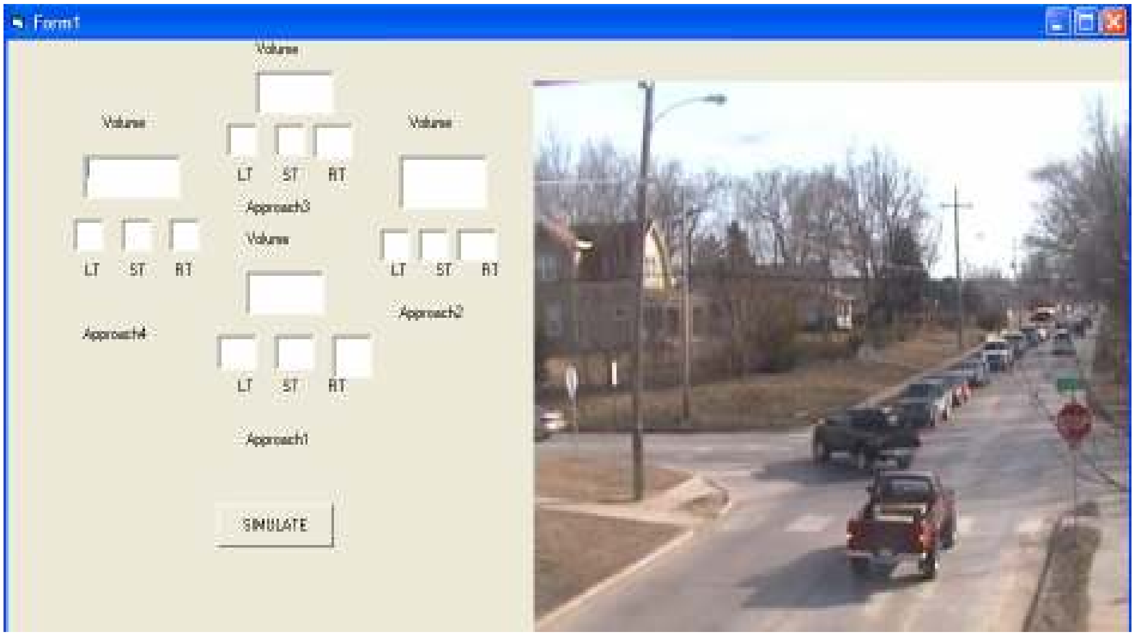


Figure 5.5 Simulation Inputs

5.4 NUMBER OF SIMULATION RUNS

Since simulation models are driven by samples of random variables from some probability distributions, simulation results may have a large variance. Tian (2002) investigated several popular simulation programs, including CORSIM, SimTraffic and VISSIM. He found out that a large number of runs were required to get stable results at near or close to capacity conditions because large variances occur. At least 20 runs were recommended when the traffic demand is near or close to the capacity.

In this study, 20 runs were conducted in the simulation and the standard deviations are shown in Figures 5.6 through 5.8. With the increase in traffic demand, a larger deviation was generally observed. As shown in Figures 5.6 and 5.7, the maximum standard error occurs when the traffic demand is at or close to the capacity, which is around 450 vph. The maximum standard error for the 95th percentile queue length was 2.5 vehicles, which is considered acceptable. The maximum standard error for average control delay was 9.3 seconds per vehicle as shown in Figure 5.7. But the standard error for service time did not show a similar trend. The variation was very small and the maximum standard error was 0.35 seconds and all the standard error was below 3%. Hence, the simulation results for the service time were deemed to be very stable. Because the control delay is a function of queue length and service time, a conclusion may be drawn that the variation of the control delay was mainly due to the variation of the queue length.

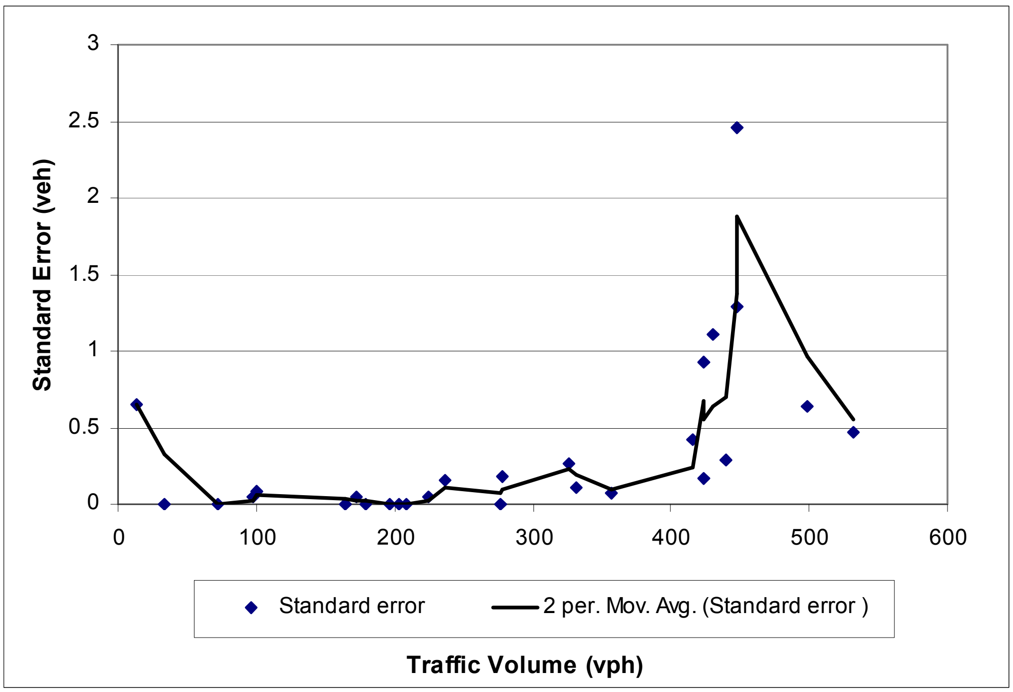


Figure 5.6 Standard Error for the 95th Percentile Queue Length

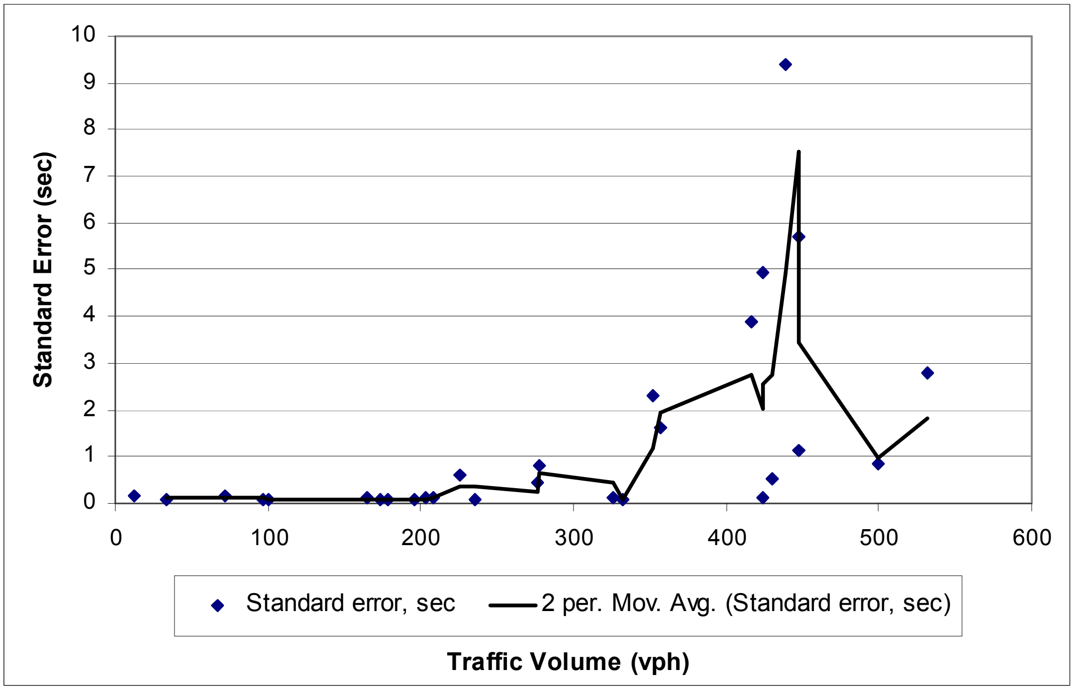


Figure 5.7 Standard Error for Average Control Delay

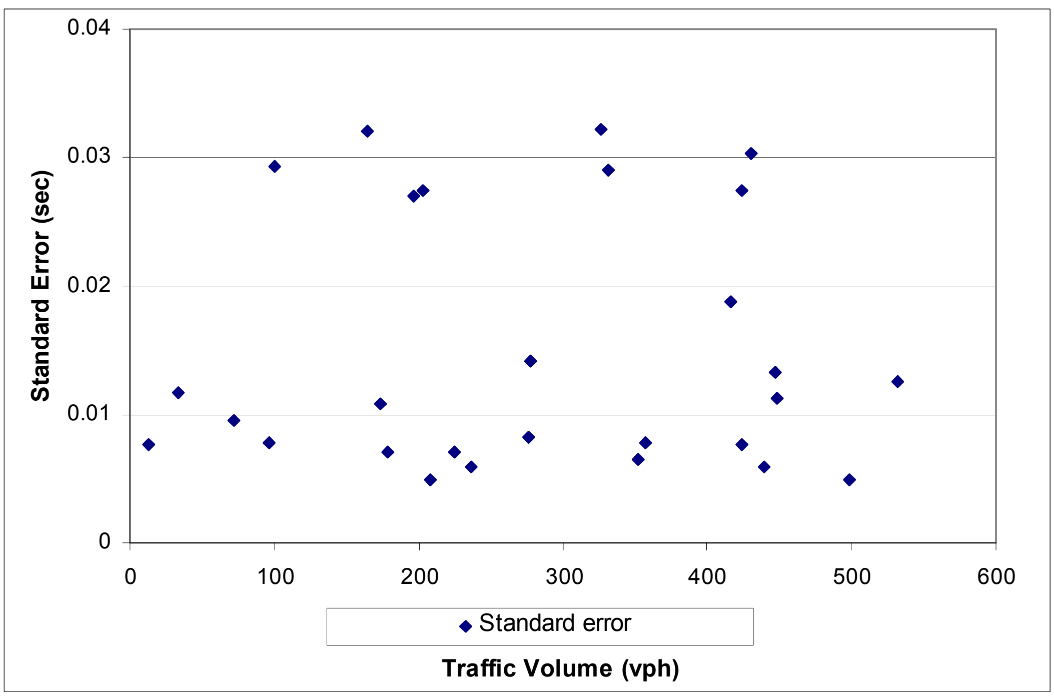


Figure 5.8 Standard Error for Service Time

5.5 SIMULATION RESULTS

5.5.1 The 95th Percentile Queue Length

The simulated 95th percentile queue length data were compared with field data and the results were presented in Figure 5.9. Because of the small sample size, two statistical parameters were used to evaluate the data sets: mean absolute error (MAE) and mean absolute percent error (MAPE) (Kyte. et al 1996), which are defined in Equation (5.1) and (5.2), respectively.

$$\text{MAE} = \frac{1}{\text{n}}\sum_{i=1}^{n}\left|d_s^i - d_f^i\right| \tag{5.1}$$

$$\text{MAPE}= \frac{1}{\text{n}}\sum_{i=1}^{n}\left|\frac{d_s^i - d_f^i}{d_f^i}\right| \tag{5.2}$$

Where,

n = the size of the sample,

d_s^i = expected values from the simulation model (veh for queue length, sec for control delay and service time) , and

d_f^i = observed values from field observation (veh for queue length, sec for control delay and service time).

As shown in Figure 5.9, a MAE of 0.55 veh and a MAPE of 0.14 were obtained based on the data sets from the simulation model and field observation. These parameters are considered good from a practical view point. It was also observed that 92% of the absolute errors between the simulation and field data sets were equal or less than one vehicle. The maximum error was three vehicles, which happened at the intersection of 15th St. and Barker Ave. The traffic on approach 1 (WB 15th Street) contained 59% right turn vehicles. Some vehicles in the second position proceeded to make right turns when the first vehicle was still at the stop line. Also, right turns were made simultaneously with vehicle on other approaches without completely stopping at the stop lines, since drivers were very familiar to the traffic situation at this intersection. There were almost no left turn conflict approach vehicles coming and only few left turn vehicles from the opposite approach.

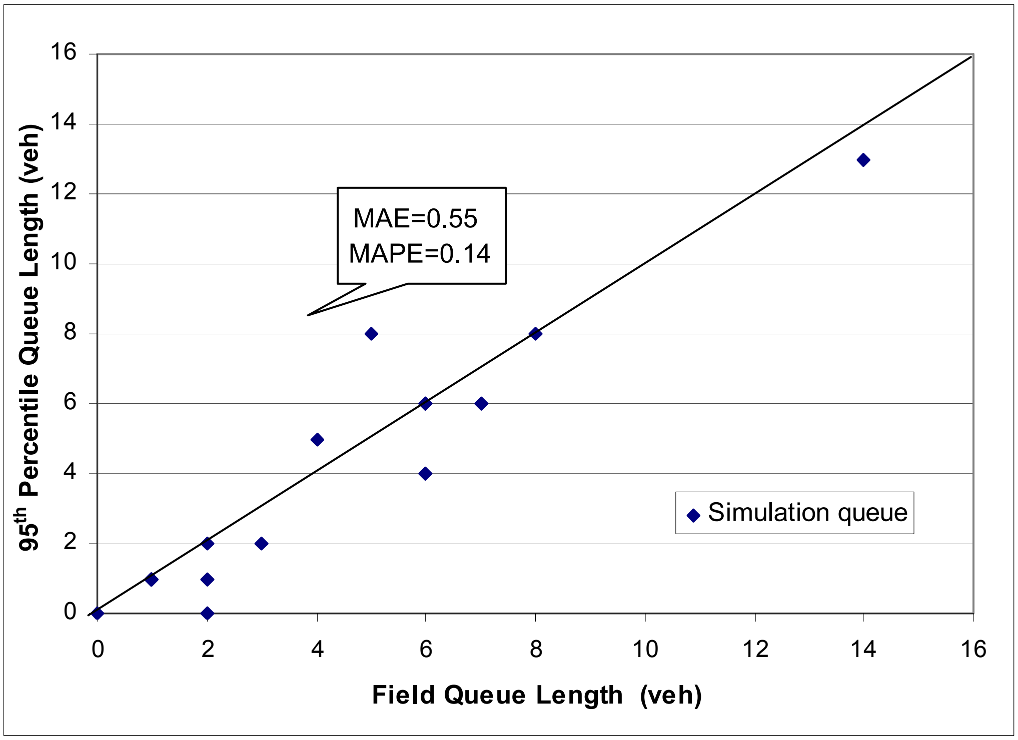


Figure 5.9 The 95th Percentile Queue Length Comparisons Between the Simulation Model and Field Data

At the same time, the simulation results were compared with the HCM 2000 model. There is no queue length model presented in the HCM 2000 manual for AWSC intersections even though it provides procedures to calculate capacities and control delays. Tian (2001) developed an empirical queue model to estimate the 95th percentile queue length based on the average queue length, which can be derived from Little's Formula based on the stop delay (the sum of queue delay and the service time). The empirical model (also referred to as the Tian model) and Little's formula are given by Equations (5.3) and (5.4), respectively.

$$L_{95\%} = 1.3L + 2.1\sqrt{L} + \frac{L}{L + 4.6} \tag{5.3}$$

Where,

$L_{95\%}$ = the 95th percentile queue length,

L = average queue length (veh) calculated from Equation (5.4).

$$L = \frac{V}{3600} \times d \tag{5.4}$$

Where,

V = traffic demand (vph),

d = average stop delay (sec)

The queue length model for TWSC intersections in the HCM 2000 was recommended for the AWSC intersections because of the same queuing theory, which can be described in the M/G2/1 model (Tian et al, 2006). In order to easily identify the models used in the following discussion, this queue length model for TWSC intersections is herein denoted as the TWSC model. The 95th percentile queue length model is shown in Equation (5.5).

$$L_{95\%} = 225\left[\frac{V}{c} - 1 + \sqrt{(\frac{V}{c} - 1)^2 + 96\frac{V}{c^2}}\right]\frac{c}{3600} \tag{5.5}$$

Where,

V = traffic demand (vph),

c = capacity of the approach (vph). It can be derived by Equation (5.6) as

$$c = \frac{3600}{h_s} = \frac{3600}{t_s + t_{mv}} \tag{5.6}$$

Where,

h_s = saturated headway (sec),

t_s = service time (sec),

t_{mv} = move-up time (sec),

As shown from Equations (5.3) thru (5.6), the average stop delay and the saturated headway data are required for the Tian model and the TWSC model, respectively, to estimate the 95th percentile queue length. In the following sections, the average stopped delay is obtained from the control delays calculated separately from the simulation model and the HCM 2000 model. The control delay is the sum of the stopped delay and the deceleration and acceleration delays. In the HCM 2000, the total deceleration and acceleration delay is 5-second, while the value of 5.74-second is used in the simulation model. The saturated headway is the sum of service time and move-up time. Service time can be obtained from the output of the simulation model and the HCM 2000 model. The 2-second move-up time is used in the HCM 2000 and the value of 3-second is used in the simulation model. Then stopped delay and saturated headway are used in the Tian model and TWSA model to estimate the 95th percentile queue length.

Based on the average stopped delay and saturated headway obtained from the simulation model, the 95th percentile queue length results predicted by the Tian model and the TWSC model are shown in Figures 5.10 and 5.11. Figure 5.10 shows that the 95th percentile queue length based on the Tian model fits the field data well, with a MAE value of 0.89 and a MAPE value of 0.30. Additionally, there is no significant difference between the queue length from the simulation model and the queue length calculated from Tian model, with a chi-square test p value of 0.90, which means there is no significant difference between the queue lengths from the simulation model and the Tian model. .

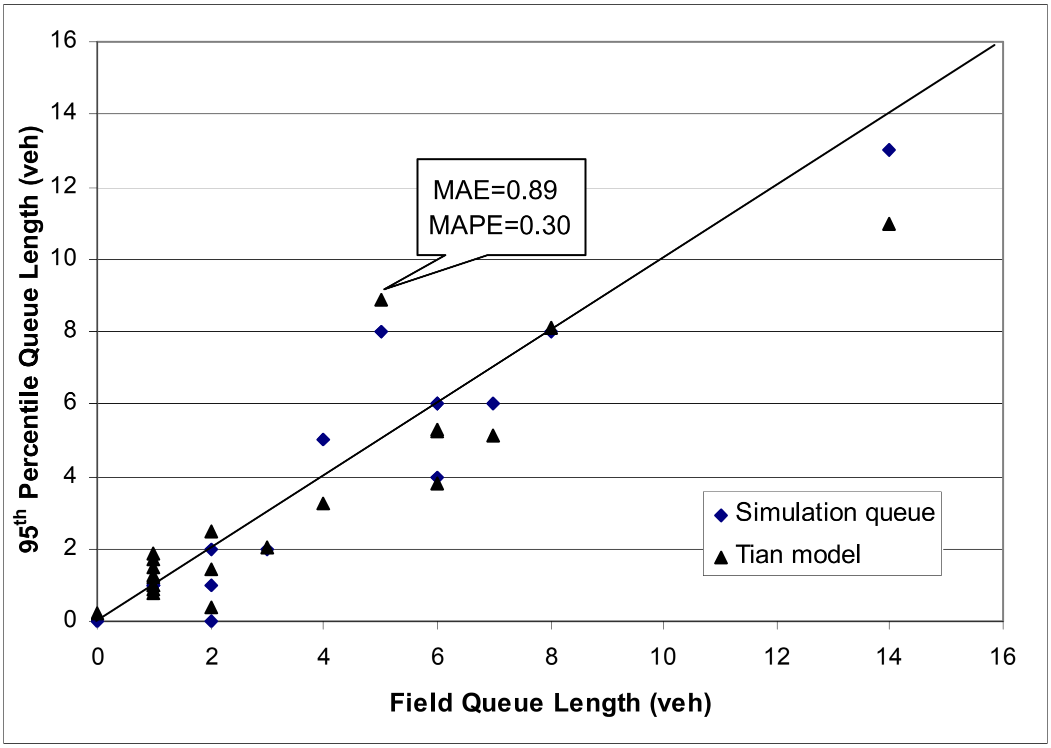


Figure 5.10 The 95th Percentile Queue Length Comparisons Between the Tian Model and Field Data, Based on Parameters from the Simulation Model

As demonstrated in Figure 5.11, it is noticed that the Tian model has a better performance when compared to the TWSC model, based on the MAE and MAPE values. Both models are considered practical in this case because the average error of estimation (MAE) is less than or close to 1.0 vehicle.

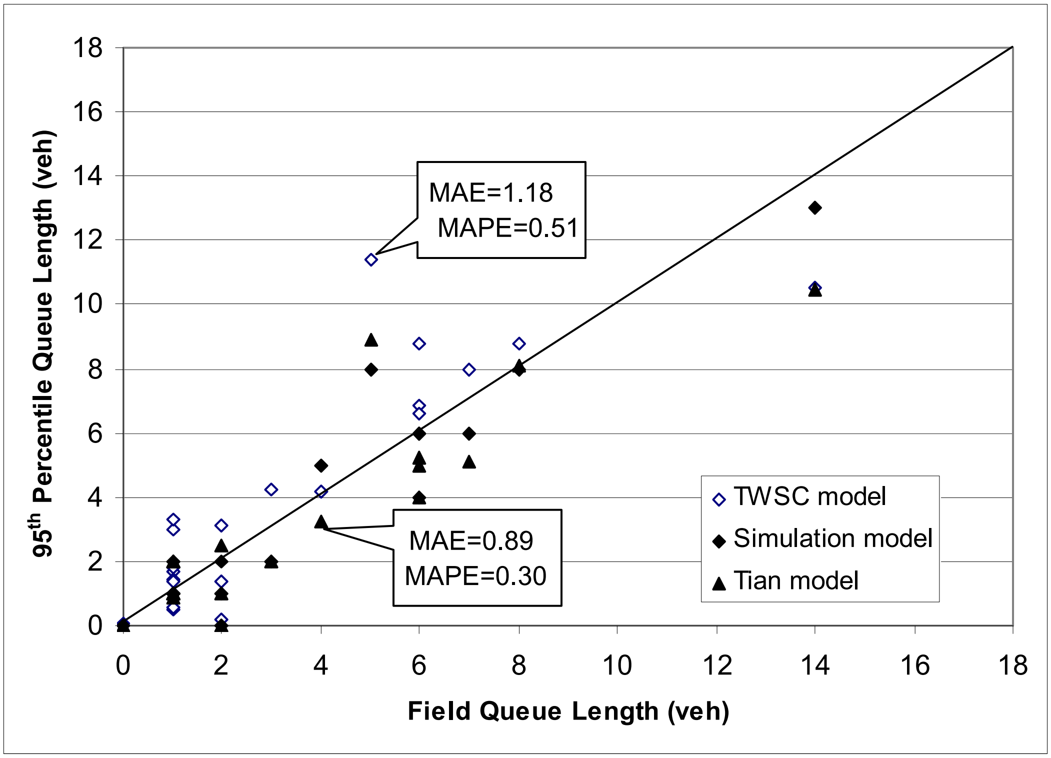


Figure 5.11 The 95th Percentile Queue Length Comparisons Between the Tian Model and Field Data, and Between the TWSC Model and Field Data, Based on Parameters from the Simulation Model

The average stopped delay and the saturated headway were obtained from the HCM 2000 model. Base on these two parameters, the 95th percentile queue length data were estimated by both the Tian model and the TWSC model, as shown in Figure 5.12. It can be clearly seen that the TWSC model predicts much better results than the Tian model. The MAE and MAPE values for the TWSC model are 1.30 and 0.36, respectively, comparing to values of 1.78 and 0.60, respectively, for the Tian model. As a result, the HCM queue length will be calculated from the TWSC model in the following sections.

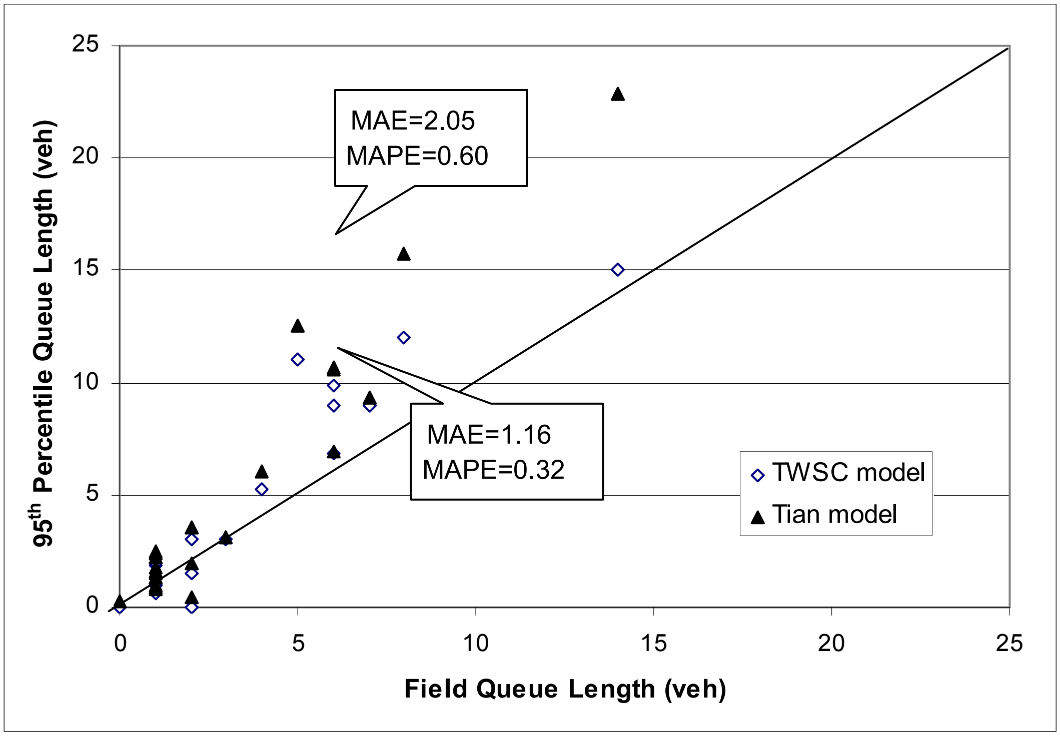


Figure 5.12 The 95th Percentile Queue Length Comparisons Between the Tian Model and Field Data, and Between the TWSC Model and Field Data, Based on Parameters from the HCM 2000 Model

Figure 5.13 compares the 95th percentile queue length between the HCM 2000 model and the simulation model. As shown in Figure 5.13, the simulation model, with a MAE value of 0.55 and a MAPE value of 0.14, predicts better results than the HCM 2000 model, which has a MAE value of 1.30 and a MAPE value of 0.36. In addition, Figure 5.13 shows that the results from the HCM 2000 model fit the field data very well when the field queue length is short. Under the situation where the field queue length is long, the HCM 2000 model predicts a longer queue than the field observations. This finding is consistent with Lai's study results. He collected queue length and volume data at 17 intersections under very similar limitations to the present study: four single-lane approaches, no bus loading nearby, minimal pedestrian activity and minimal platoon impacts. The queue lengths from the HCM 2000 model and the field observations were compared. Figure 5.14 shows the 95th percentile queue length comparisons between the HCM

2000 model and the field data from Lai's study. It is noticed that when the volume/capacity ratio is small, the HCM 2000 model predicts very accurate queue length. But when volume/capacity ratio is larger than 0.48, 77% of the computed queue lengths from the HCM 2000 model are longer than those observed in the field.

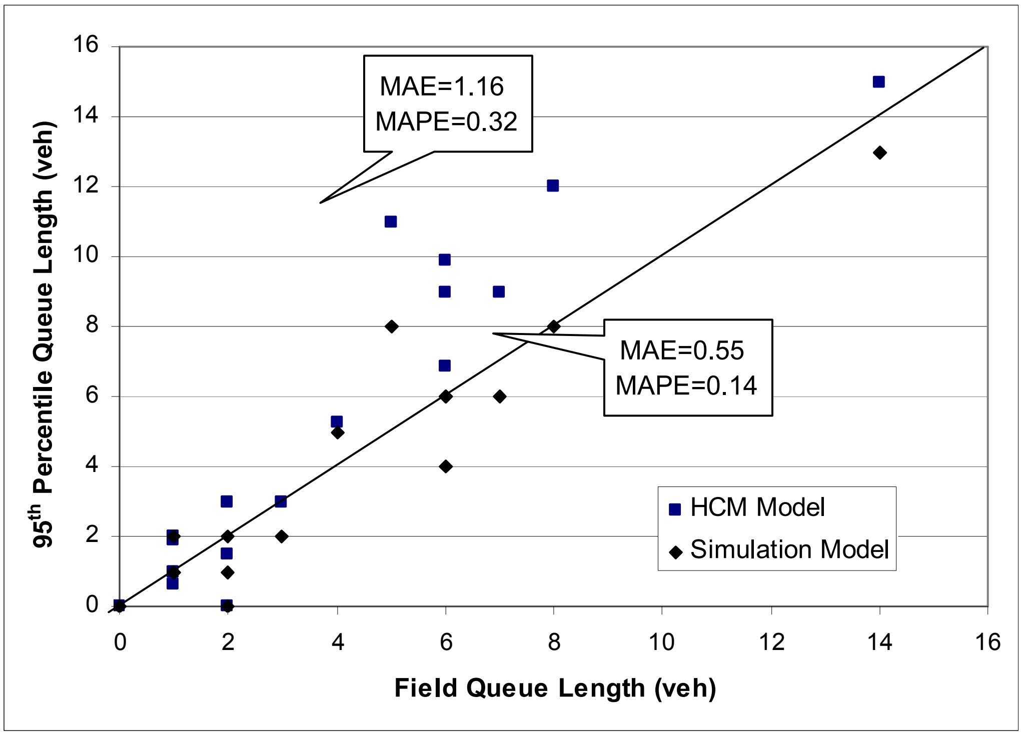


Figure 5.13 The 95th Percentile Queue Length Comparisons between the HCM 2000 Model and the Simulation Model

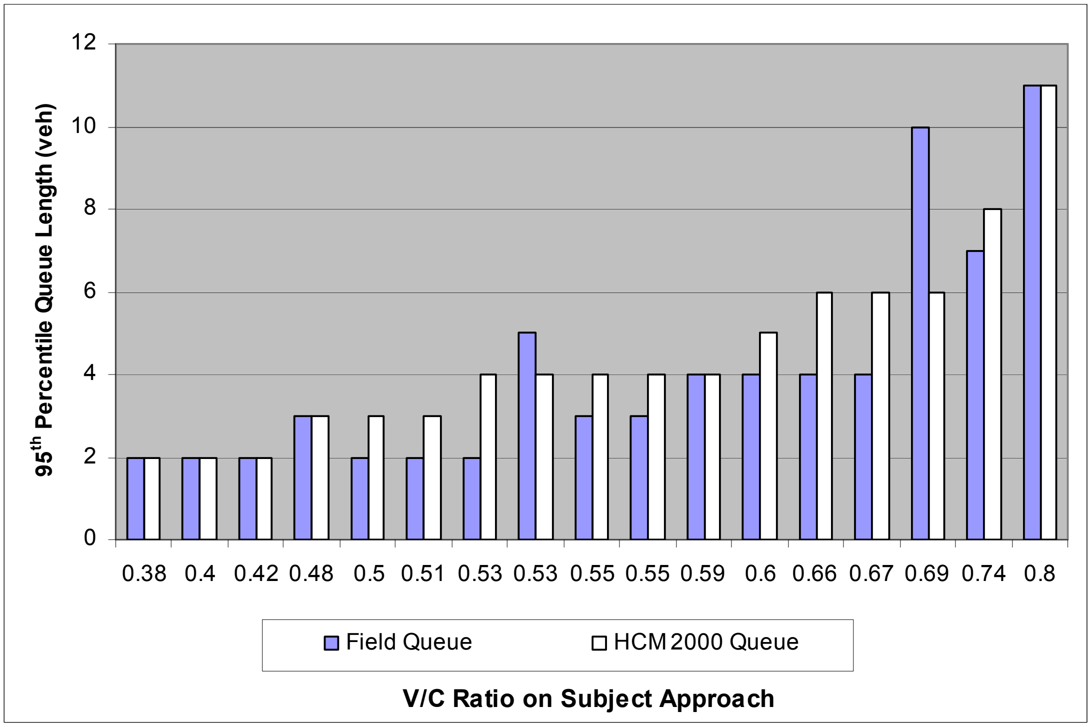


Figure 5.14 The 95th Percentile Queue Length Results from Lai's Study (2009)

5.5.2 Control Delay and Service Time

Control delay data were collected at only one intersection: 11th Street and Connecticut Street in Lawrence. The field data and the results based on both the simulation model and the HCM 2000 model are tabulated in Table 5.1. As shown in Table 5.1, in general, the HCM 2000 model predicts longer control delay values than those observed in the field observation data. It can be seen that the average control delay predicted by the simulation model fits the field data much better than those predicted by the HCM 2000 model. A MAE value of 2.45 and a MAPE value of 0.13 are observed for the simulation model, while the HCM 2000 model has MAE and MAPE values of 6.49 and 0.37, respectively.

Table 5.1 Control Delay Based on Field Observation, the HCM 2000 Model and the Simulation Model

Control delay at intersection: 11th Street and Connecticut Street						
	Approach 1	Approach 2	Approach 3	Approach 4	MAE	MAPE
HCM	37.17	14.09	26.8	14.75	6.49	0.37
Simulation	19.42	11.2	15.35	10.81	2.45	0.13
Field data	26.5	12.1	17.99	10.35		

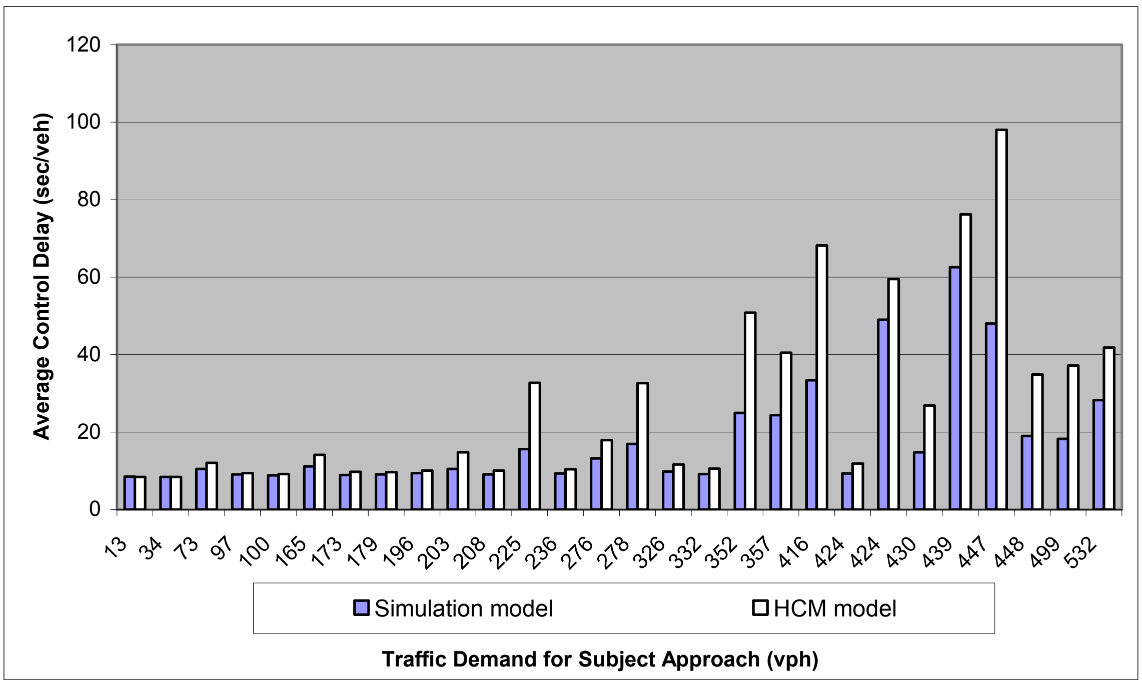


Figure 5.15 Average Control Delay Comparisons Between the HCM 2000 Model and the Simulation Model

For the six selected intersections in Chapter 4, the average control delay was predicted using both the HCM 2000 model and the simulation model. Figure 5.15 shows the average control delay results based on both models. As shown in Figure 5.15, the average control delay data predicted by the two models are very close when the traffic demands are low. With increasing traffic demands, the HCM 2000 model predicts longer control delay than the simulation model.

Since it is easy to collect service time data from the video data processing program, service time data were collected at all six selected intersections. Figures 5.16(a) and 5.16(b) shows the results from the HCM 2000 model, the simulation model, and the field observation. Again, the results predicted by the simulated model fit the observed data better with a MAE value of 0.42 and a MAPE value of 0.13, compared to a MAE value of 0.96 and a MAPE value of 0.29 for the results based on the HCM 2000 model. The HCM 2000 model predicts a longer service time than the simulation model, and this observation is consistent with longer queues and longer delays predicted by the HCM 2000 model. The longer service time suggests a shorter move-up time used in the HCM 2000 model. The 2–second move-up time used in the HCM is shorter than the actual move-up time taken for vehicles to move to the stop line from the second position.

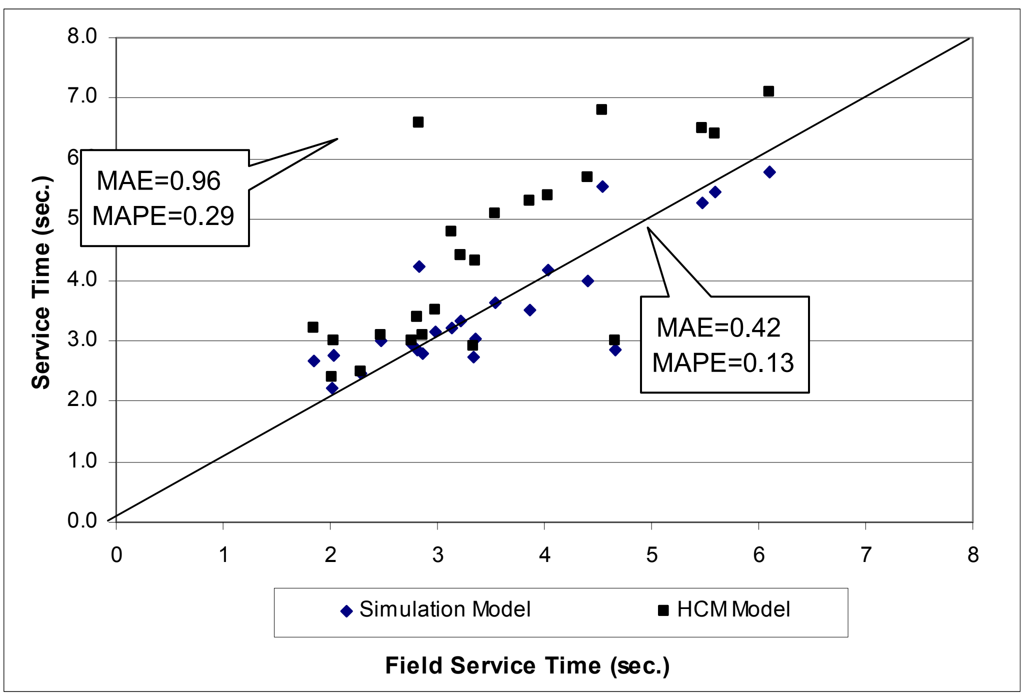


Figure 5.16(a) Service Time Comparisons Between the HCM 2000 model and the Simulation Model

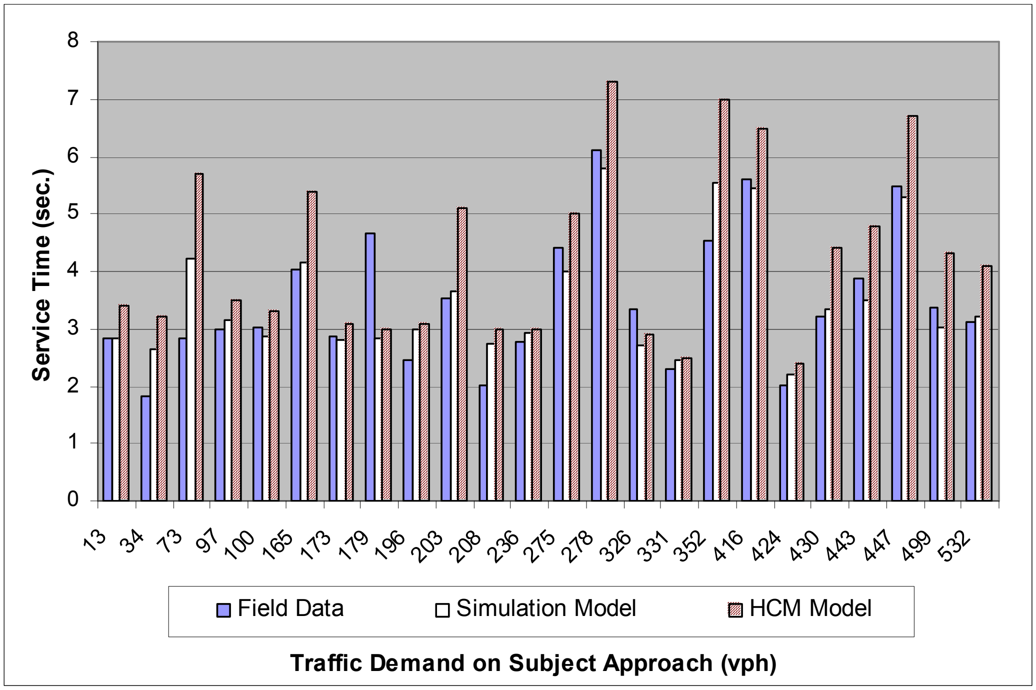


Figure 5.16(b) Service Time Comparisons Between the HCM 2000 model and the Simulation Model

5.5.3 Traffic Operation Characteristics at FWSC Intersections

Simulations were conducted to investigate the relationship between average control delays and traffic demands, with different volume ratios between minor streets and major streets. The procedure involves changing the volumes on the minor street based on volume ratios from 0.1 to 0.5 under traffic demands between 300 vph to 800 vph on the major street. As shown in Figure 5.2, approach 1 and approach 3 are on the major street and thus the volumes for approach 1 and approach 3 are denoted as “major volume.” Figures 5.17 and 5.18 show the average control delay results from the simulation model and the HCM 2000 model, respectively.

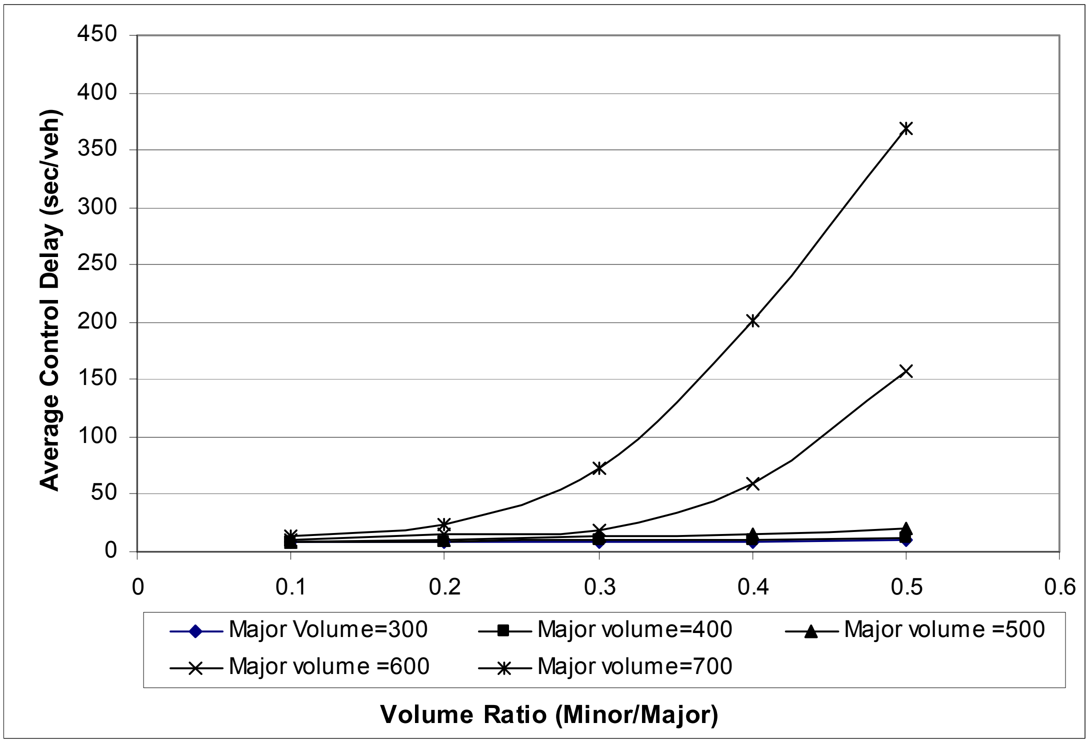


Figure 5.17 Average Control Delay on Major Street Under Different Traffic Volumes Based on the Simulation Model

Figures 5.17 and 5.18 indicate that for both the simulation model and the HCM 2000 model, average control delay increases as the major volume and the volume ratio between minor streets and major streets increases. When the volumes on the major street are less than 400 vph, the average control delay for a certain major volume condition is well below 30 sec/veh and remains almost constant as volume ratio increases. When the major volume is greater than 400 vph, the average control delay demonstrates an exponential increase with increasing volume ratios.

On the other hand, some differences are observed between the results based on the simulation model and the HCM 2000 model. First of all, when the volume on the major street is 500 vph, the average control delays, based on the HCM 2000 model, have an obvious increase with increasing volume ratios between the minor street and the major street. But for the simulation model, the control delay increase is not that dramatic as the volume ratios between the minor street and the major street increase. Secondly, for the simulation model, no significant difference is observed for the average control delays at different major volumes when the volume ratio between the minor street and the major street is 0.1. However, at the volume ratio of 0.1, the average control delays for different major volumes vary significantly based on the HCM 2000 model.

Another difference between these two models is the value of the threshold volume ratio or critical point (exponential break point on the graph, Oricchio, 2007). For the simulation model, a volume ratio of 0.2 is observed as the critical point. The average control delay increases slowly before the critical point and has an exponent increase after passing it. But the HCM 2000 model shows an exponential increase when the volume ratio is 0.1.

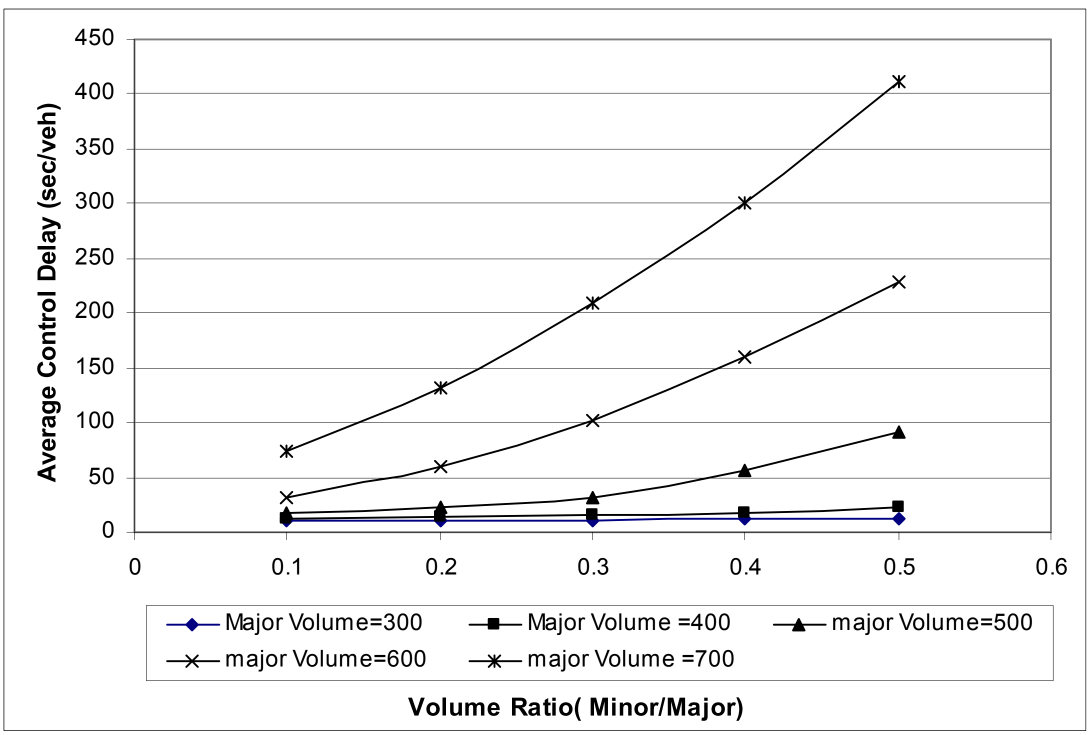


Figure 5.18 Average Control Delay on Major Street Under Different Traffic Volumes Based on the HCM 2000 Model

Figure 5.19 shows that the average control delays increase exponentially with increasing traffic demands on the major street when the volume ratio is 0.5. It can be clearly seen that the critical volumes that cause the average control delay to behave exponentially for the two models are quite different. The critical volume is 500 vph for the simulation model and 400 vph for the HCM 2000 model. In addition, the two curves show trends similar to each other after the critical points.

Figure 5.20 shows the control delay comparison between the two models when the major volume is 700 vph. The two models demonstrate a similar trend that the average control delays increase exponentially with increasing volume ratios. This finding contradicts with Oricchio's simulation results as shown in Figure 5.21. In his research, a model was programmed using VB and VISSIM COM to simulate the traffic at intersections under red flasher control for both streets, which functions similar to a FWSC intersection. Figure 5.21 shows that the average control delays remain nearly constant under different volume ratios.

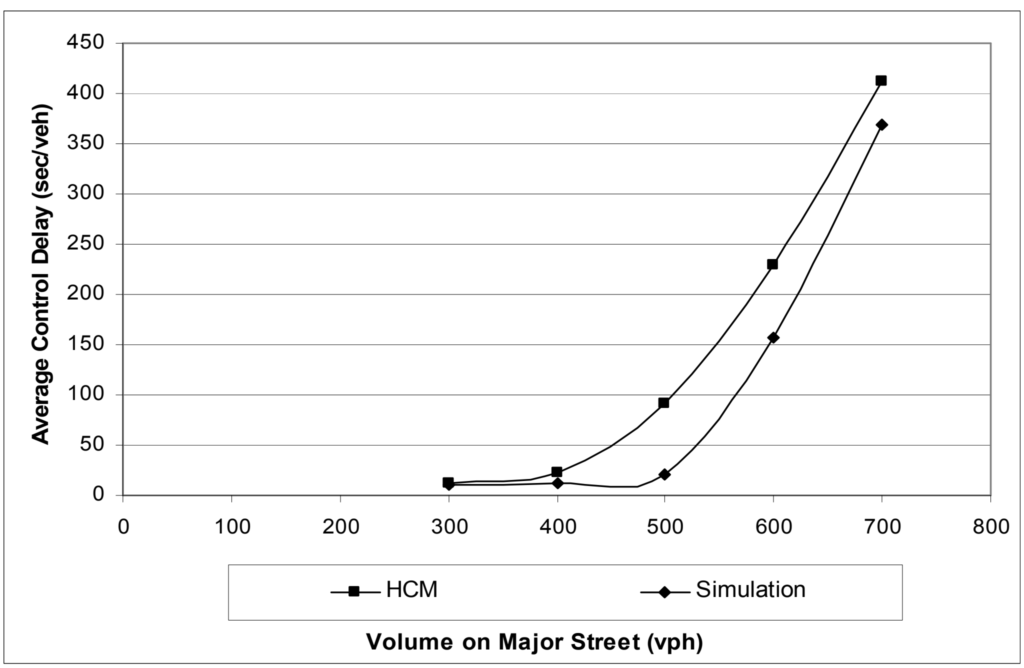


Figure 5.19 Average Control Delay on Major Street with a Volume Ratio of 0.5, Based on the Simulation and HCM 2000 Models

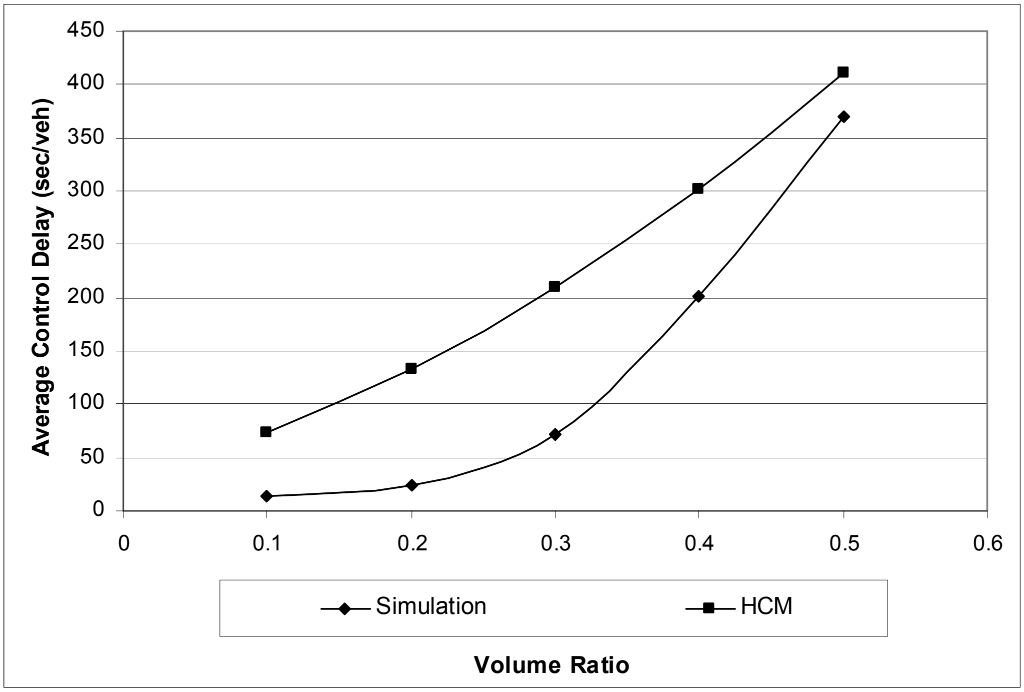


Figure 5.20 Average Control Delay on Major Street with a Major Volume of 700 vph, Based on the Simulation and the HCM 2000 Model

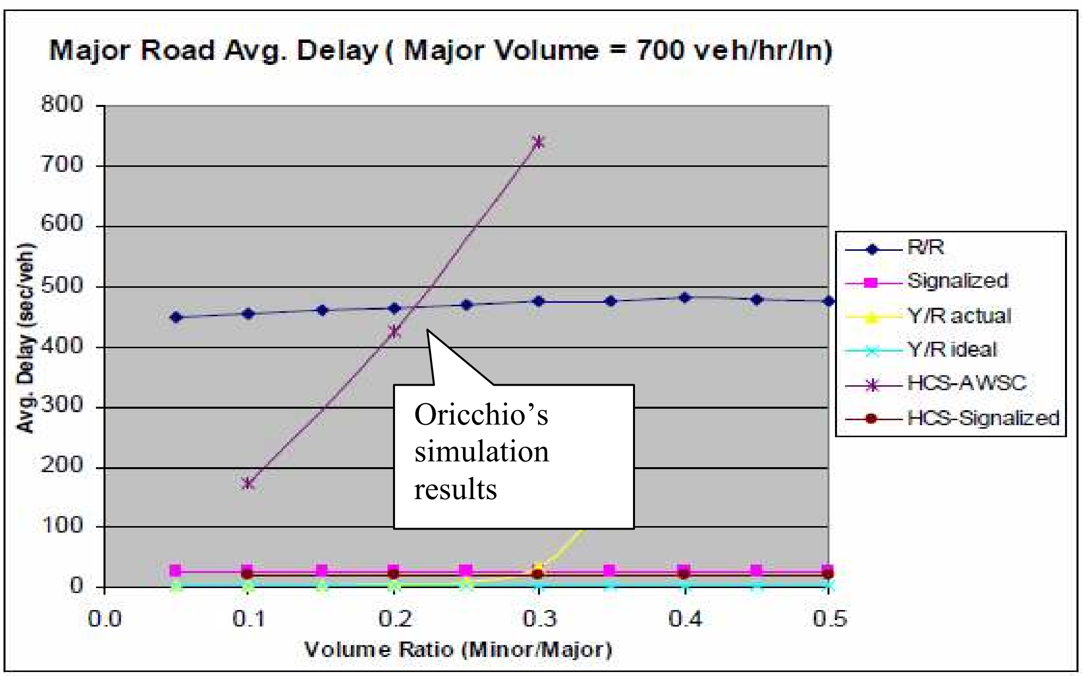


Figure 5.21 Average Control Delay on Major Street with a Major Volume of 700 vph based on Oricchio's Simulation (Oricchio, 2007)

Figures 5.22 and 5.23 show the 95th percentile queue length data from the simulation model and the HCM 2000 model. It is obvious that both models predict exponential increase for the queue length with increasing traffic demand on the major street at given volume ratios between the minor street and the major street. But the simulation model shows a rapid increase when the volume is greater than 500 vph on the major street, while the HCM 2000 model has a critical volume of 400 vph. As volume ratios increase from 0.1 to 0.5 under any given volume on the major street, the simulation model demonstrates very obvious exponential increases, while the HCM 2000 model shows a roughly linear increase. Figure 5.24 shows the increasing curves for both the simulation and HCM 2000 models. It can be concluded that the 95th percentile queue length based on the HCM 2000 model increases evenly with every 10% of increasing volumes on the minor street. For example, when the major volume is 700 vph, the 95th percentile queue length increases by about seven veh. with every 10% of increase in traffic demand on the minor street. Additionally, the 95th percentile queue length based on the simulation model is much longer than that predicted by the HCM 2000 model even though the HCM 2000 model predicts longer average control delay than the simulation model.

For the HCM 2000 model, a queue length of 46 vehicles at a volume ratio of 0.5 (as shown in Figure 5.25) seems too short for the average control delay of 411 seconds (as shown in Figure 5.19) when the major volume is 700 vph. If Little's formula and the Tian's model are used to calculate the 95th percentile queue length, a queue length of 122 vehicles is predicted and this is very close to the queue length of 120 vehicles predicted by the simulation model. There may be two factors contributing to the shorter queue length but longer control delay predicted by the HCM 2000 model compared to the queue length and control delay from the simulation model. One factor is the longer service time caused by the short move-up time. The other factor is the TWSC model that was used to calculate the HCM queue length. The TWSC model may be practical under certain traffic demand ranges, but apparently not for the situation when volumes are close to intersection capacity.

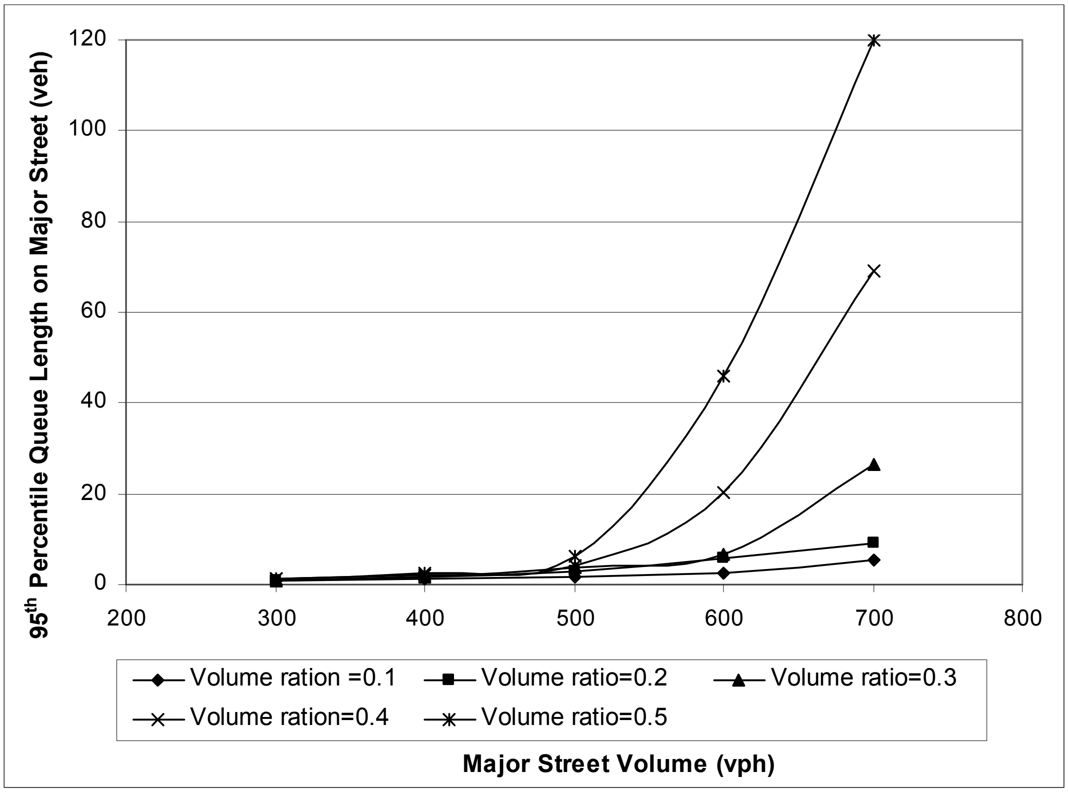


Figure 5.22 The 95th Percentile Queue Length for the Major Street at Different Volume Ratios, Based on the Simulation Model

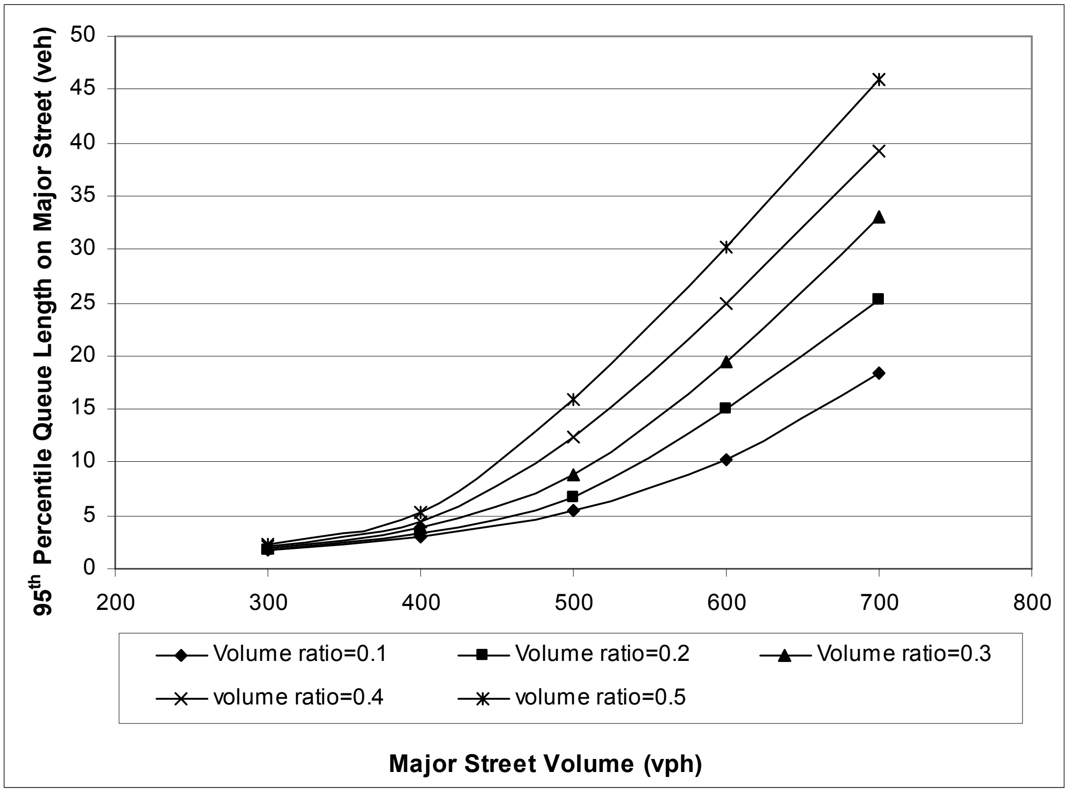


Figure 5.23 The 95th Percentile Queue Length for the Major Street at Different Volume Ratios, Based on the HCM 2000 Model

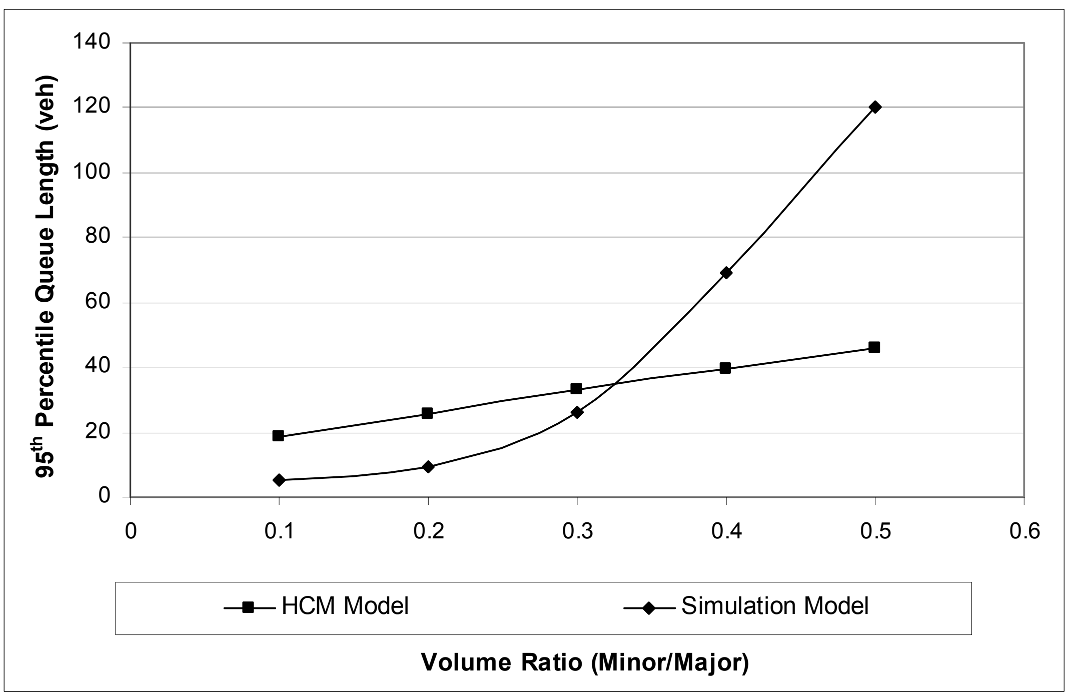


Figure 5.24 The 95th Percentile Queue Length for the Major Street with a Traffic Demand of 700 vph on the Major Street, Based on the Simulation Model and HCM 2000 Model

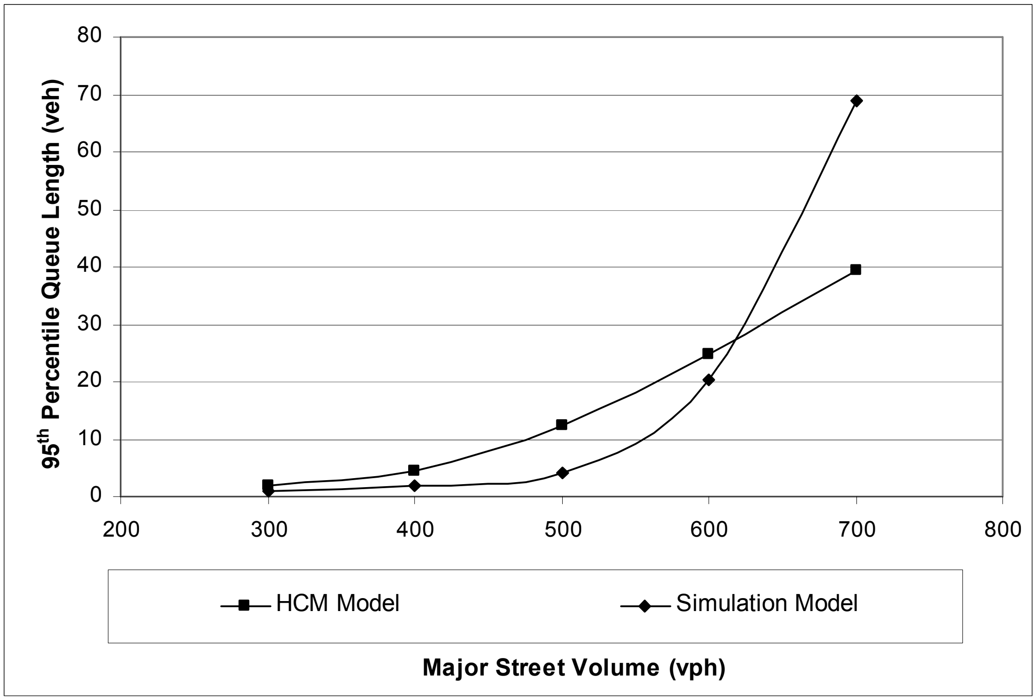


Figure 5.25 The 95th Percentile Queue Length for the Major Street with a Volume Ratio of 0.4, Based on the Simulation Model and the HCM 2000 Model

Major street service time values from the simulation model are shown in Figure 5.26, which clearly shows that the service time follows a similar increasing pattern as the average control delay. Exponential increases in the service time are observed when the volume ratio between the minor street and the major street is greater than 0.3. Figure 5.27 shows the relationship between calculated service time from the HCM 2000 model and the traffic demand on the major street. A quite different pattern is observed because the service time increases rapidly when the volumes on the major street are less than 500 vph. Then it increases at a slightly lower rate as the traffic demand on the major street increases.

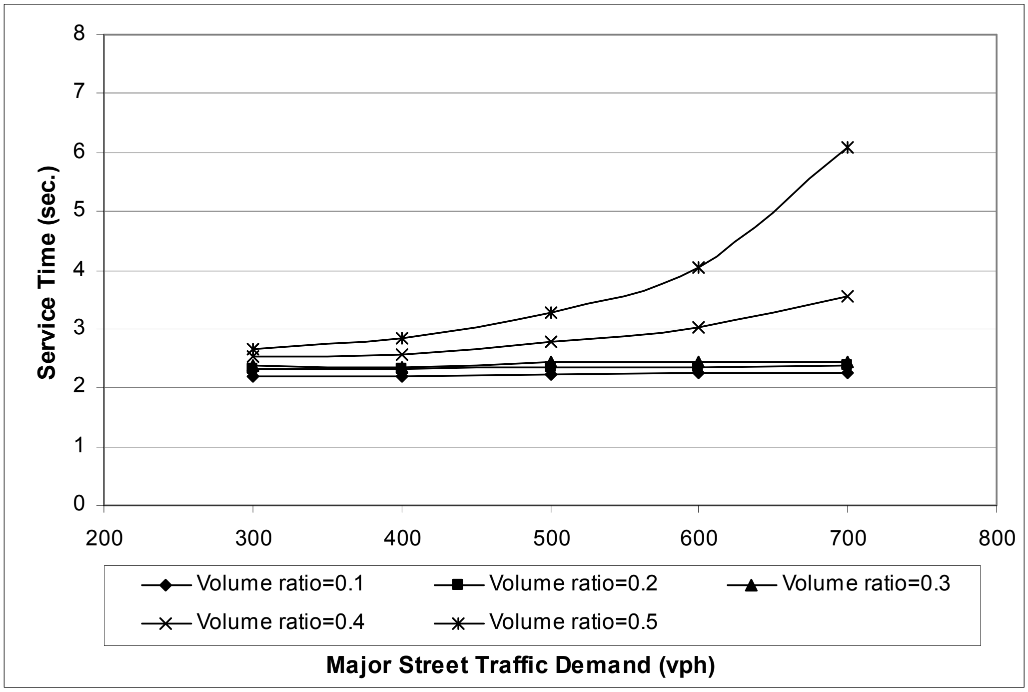


Figure 5.26 Service Time for the Major Street at Different Volume Ratios, Based on the Simulation Model

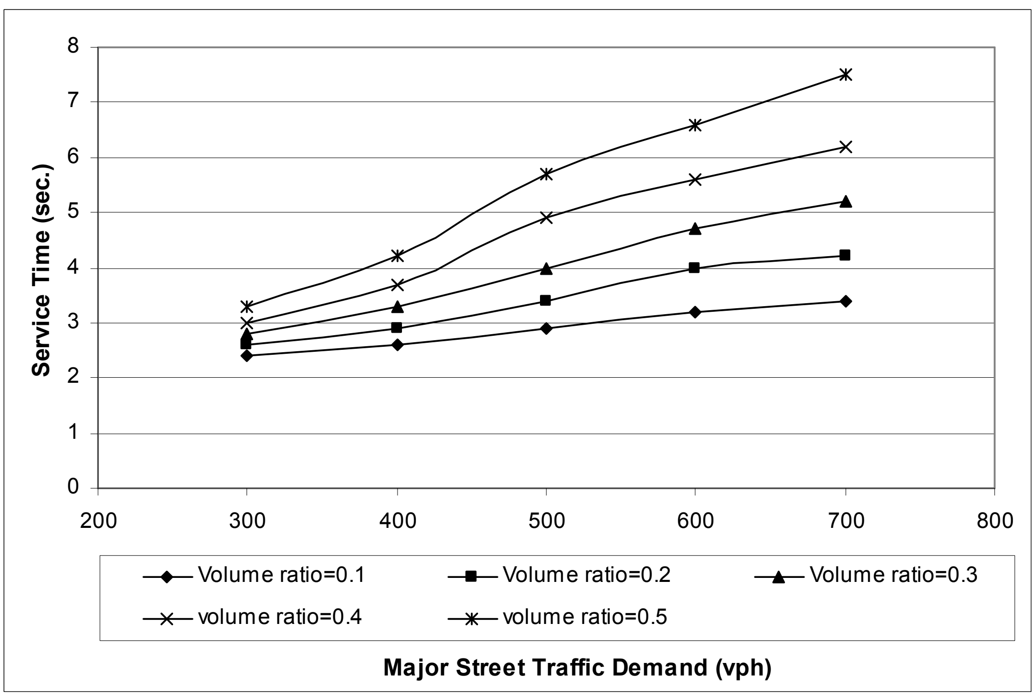


Figure 5.27 Service Time for the Major Street at Different Volume Ratios, Based on the HCM 2000 Model

Figures 5.28 and 5.29 present the comparisons of service time values based on the simulation model and the HCM 2000 model under certain traffic conditions. Figure 5.28 shows the service time comparisons at a volume ratio of 0.5. Several differences between the two models are observed. First, the service time predicted by the HCM 2000 model is higher than that based on the simulation model. Different move-up times used in the two models may contribute this difference. A 2-second move-up time is used in the HCM 2000 model while a 3-second move-up time is used in the simulation model. It was mentioned earlier that a 2-second move-up time may be short compared to the data collected in the field. Secondly, both models show that the service time follows a transverse curve with increasing volumes at a volume ratio of 0.5 between the minor street and the major street. However, the turning point for the HCM 2000 model occurs when the major volume is at 500 vph instead of 600 vph, where the turning point for the simulation model occurs. As shown in Figure 5.29, the service time values based on the two models have different increasing paths with increasing traffic demands on the minor street, when the major volume is 700 vph. The service time based on the HCM 2000 model shows a roughly linear increase, while the simulation model predicts a transverse curve increase for the service time.

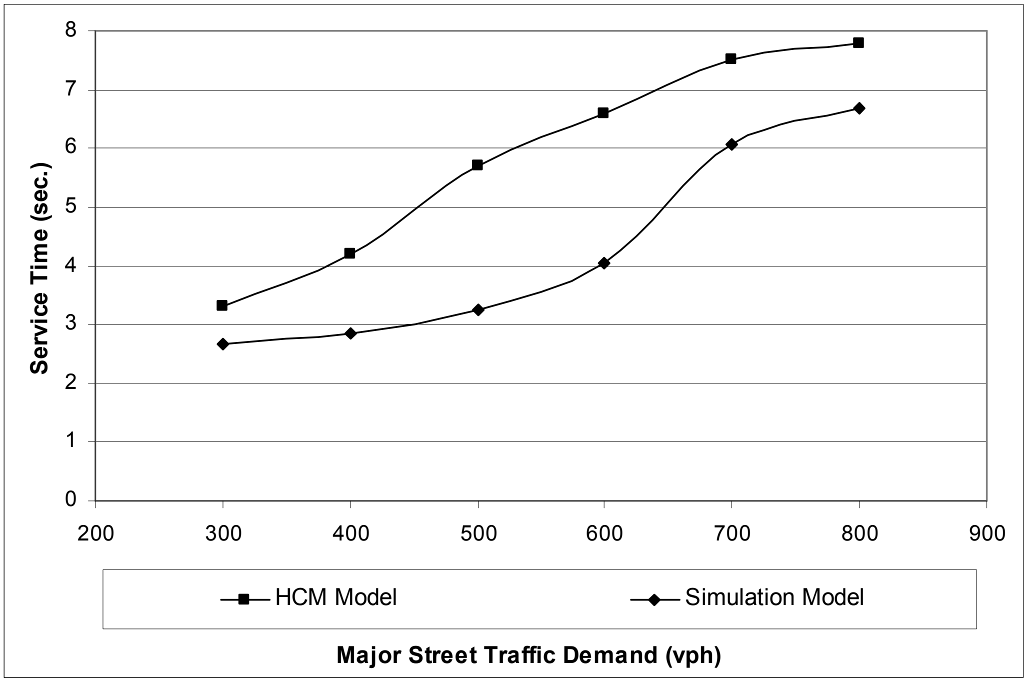


Figure 5.28 Service Time on the Major Street at a Volume Ratio of 0.5, Based on the Simulation Model and the HCM 2000 Model

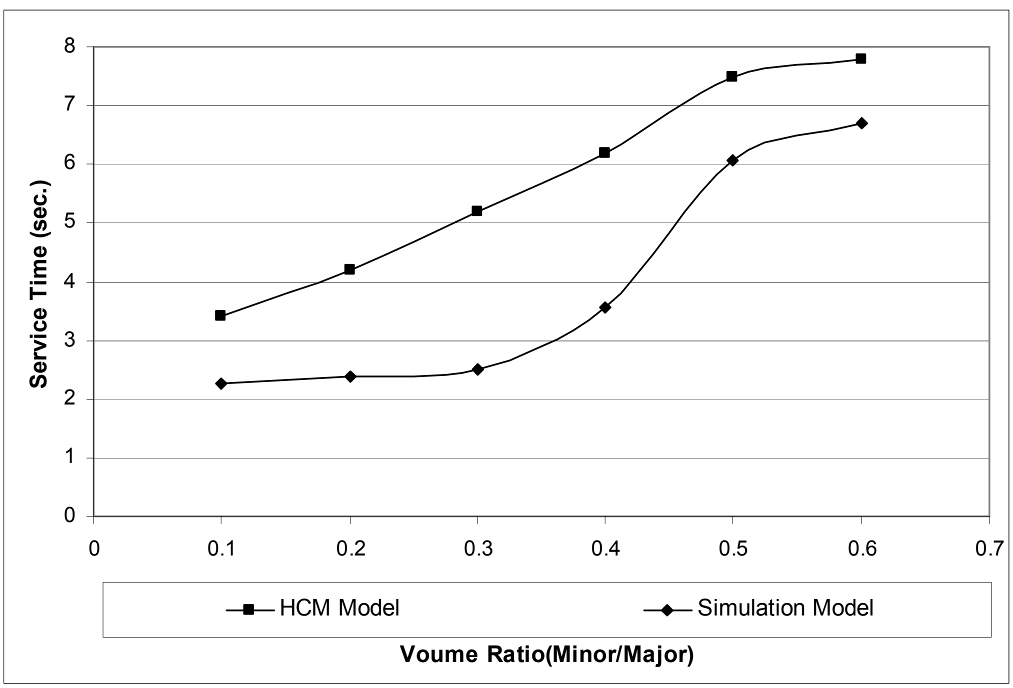


Figure 5.29 Service Time on the Major Street with Volume of 700 vph on the Major Street, Based on the Simulation Model and the HCM 2000 Model

5.5.4 Effects of Platoon Arrival on Queue Length, Average Control Delay, and Service Time

The Cowan's M3 headway distribution model was used in the simulation model to generate various headways. The equations were presented in Chapter 4. In that formula, b is the bunching factor, which indicates the degree of platoon arrival conditions. When the bunching factor b is zero, the M3 model becomes a shifted negative exponential distribution. Most FWSC intersections in this research are located far from signalized intersections and thus, a heavy platoon arrival rate is not common. In this study, three different b values, namely 0, 0.6, and 1.5, were used to investigate the effects of platoon arrival through simulations.

Two assumptions were made to perform the simulations. While increasing the traffic demand on a subject approach, the volumes on other approaches were kept the same, which was 300 vph. The other assumption was that all the traffic was through traffic. Figures 5.30 through 5.32 present the simulation results. Simulation results indicate that as platoon arrivals increase, both the average control delay and the 95th percentile queue length increase as well. However, as shown in Figure 5.32, the platoon arrival has no significant impact on service time because all vehicles are required to stop completely at the stop line. It was also noticed that when the subject approach volume was close to 600 vph, the service time kept constant, which was the saturated service time. In another word, the system reached its capacity with the traffic demand.

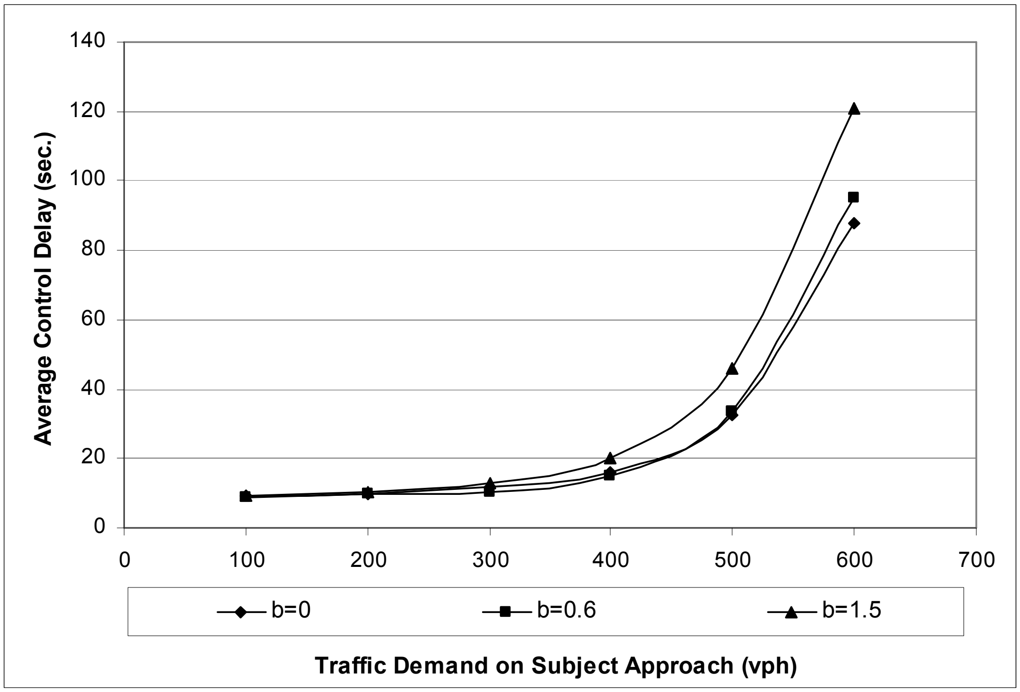


Figure 5.30 Effects of Platoon Arrival on Average Control Delay, Based on the Simulation Model

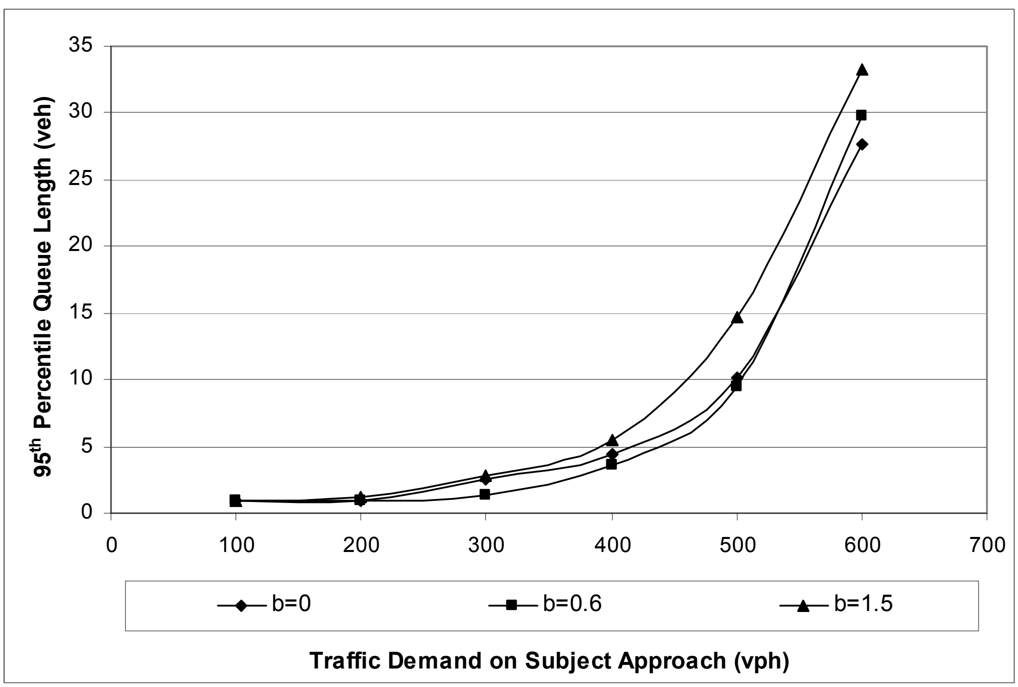


Figure 5.31 Effects of Platoon Arrival on 95th Percentile Queue Length, Based on the Simulation Model

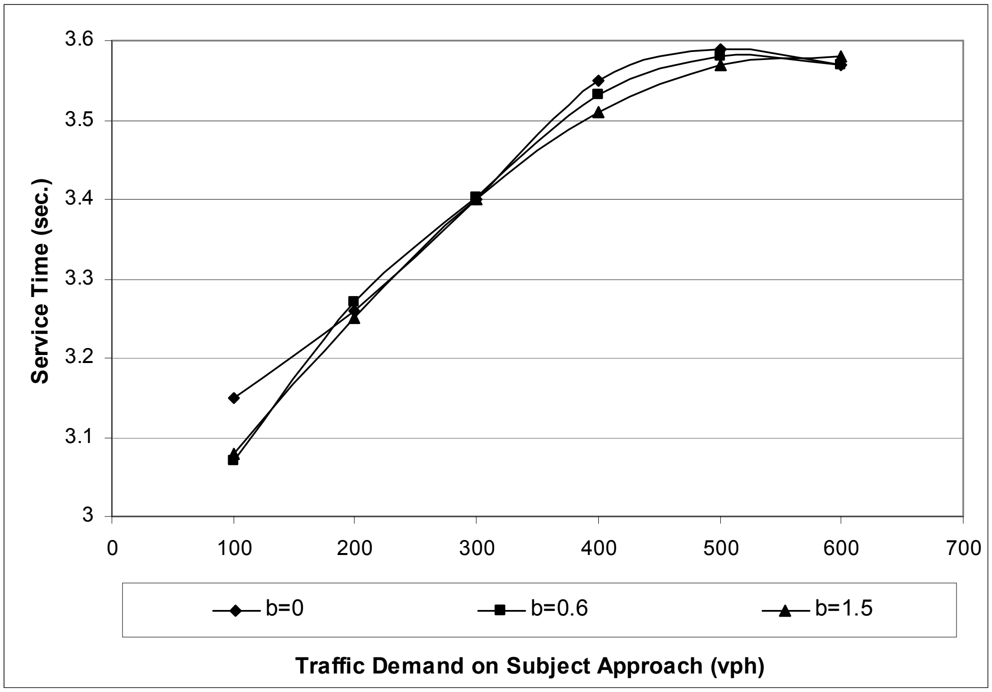


Figure 5.32 Effects of Platoon Arrival on Service Time, Based on the Simulation Model

5.6 SIMULATION DATA VALIDATION

Simulation model validation is to use field data that were not included in the calibration to test the calibrated simulation model. Most of the collected data at the six FWSC intersections were used to develop the simulation model. In order to validate the simulation model, the field data were collected at the following intersections: W 27th St at Alabama St (Lawrence, KS), Harvard Rd at Crestline Dr. (Lawrence, KS) and W 87 St. at Lamar Ave.(Overland park, KS).

5.6.1 Intersection locations and volume data

Table 5.2 Peak Hour Factors for the Four Approaches (27St. and Alabama St.)

NB	WB	SB	EB
0.69	0.9	0.86	0.77

The intersection of 27St. and Alabama St., shown in Figure 5.33, is located in a neighborhood in the southeast area of Lawrence (KS). It is a single lane FWSC intersection with no parking on the streets, which meets the requirements of this study. Tables 5.2 and 5.3 show the peak hour factors for each approach and volumes for the approaches and the intersection.

Figure 5.33 FWSC Intersection 27 St. and Alabama (Lawrence, KS)

Table 5.3 Peak Hour Volumes for the Intersection (27St. and Alabama St.)

Time Interval	NB Volume	WB Volume	SB Volume	EB Volume	Intersection Volume
4:45-5:00 PM	2	37	33	34	106
5:00-5:15 PM	8	35	31	52	126
5:15-5:30 PM	6	40	28	44	118
5:30-5:45 PM	6	43	22	31	102
Approach Volume	22	155	114	161	∑=452

Figure 5.34 FWSC Intersection Harvard Rd. and Crestline Dr. (Lawrence, KS)

The intersection of Harvard Rd at Crestline Dr., shown in Figure 5.34, is located on the west side of Lawrence. No parking is allowed on either street. There is a school in the northwest corner of the intersection. Since data were collected during the summer time, there were only few pedestrians during the data collection time. Table 5.4 and Table 5.5 show the peak hour factors for each approach and the volumes for the approaches and the intersection.

Table 5.4 Peak Hour Factors for the Four Approaches (Harvard Rd. and Crestline Dr.)

EB	NB	WB	SB
0.67	0.81	0.81	0.88

Table 5.5 Peak Hour Volumes for the Intersection (Harvard Rd. and Crestline Dr.)

Time Interval	EB Volume	NB Volume	WB Volume	SB Volume	Intersection Volume
4:33-4:48 PM	1	42	13	25	81
4:48-5:03 PM	2	50	22	26	100
5;03-5:18 PM	3	66	21	32	122
5:18-5:33 PM	2	57	15	29	103
Approach Volume	8	215	71	112	∑=406

The third intersection used for the validation is the intersection of 87St. and Lamar Ave. located in Overland Park (37 miles east from Lawrence), Kansas, which is the second most populous city in Kansas with a population of 173,250. The intersection meets all the requirements for the project scope. Table 5.6 and Table 5.7 present the peak hour factors for each approach and volumes for the approaches and the intersection.

Table 5.6 Peak Hour Factors for the Four Approaches (87 St. and Lamar Ave.)

SB	EB	NB	WB
0.79	0.72	0.81	0.76

Figure 5.35 FWSC Intersection 87 St. and Lamar Ave. (Overland Park, KS)

Table 5.7 Peak Hour Volumes for the Intersection (87 St. and Lamar Ave.)

Time Interval	SB Volume	EB Volume	NB Volume	WB Volume	Intersection Volume
5:00-5:15 PM	76	49	79	21	225
5:15-5:30 PM	108	42	66	33	249
5:30-5:45 PM	81	65	67	24	237
5:45-5:60 PM	75	31	43	22	171
Approach	340	187	255	100	∑=882

5.6.2 Queue Length Validation

The TWSC model was used to calculate the 95th percentile queue length for the HCM 2000 model. The 95th percentile queue length comparisons between the HCM 2000 model and the simulation model for the three intersections are listed in Table 5.8 through Table 5.10. Both models predict reasonable results. The maximum error for the HCM 2000 model is 2 veh while the simulation mode1 has a maximum error of 1 veh. Overall, the simulation model predicts more accurate queue lengths than the HCM 2000 model, with an average estimate error of 0.17 veh and an absolute percent error of 0.06. The HCM 2000 has a MAE value of 0.5 veh and a MAPE value of 0.24 (See Figure 5.36).

Table 5.8 95th Percentile Queue Length Comparisons Between the HCM Model and the Field Data and between the Simulation Model and the Field Data (27 St. and Alabama St.)

27St. and Alabama St.		NB	WB	SB	EB
FIELD	95th Queue Length	0	1	1	2
HCM Model	95th Queue Length	0	1	1	1
	Error, Veh	0	0	0	1
Simulation Model	95th Queue Length	0	1	1	1
	Error, Veh	0	0	0	1

Table 5.9 95th Percentile Queue Length Comparisons Between the HCM Model and the Field Data and between the Simulation Model and the Field Data (Harvard Rd. and Crestline Dr.)

Harvard Rd.. and Crestline Dr.		EB	NB	WB	SB
FIELD	95th Queue Length	0	2	1	1
HCM Model	95th Queue Length	0	1	0	1
	Error, Veh	0	1	1	0
Simulation Model	95th Queue Length	0	2	1	1
	Error, Veh	0	0	0	0

Table 5.10 95th Percentile Queue Length Comparisons Between the HCM Model and the Field Data and between the Simulation Model and the Field Data (87St. and Lamar Ave.)

87St. and Lamar Ave.		SB	EB	NB	WB
FIELD	95th Queue Length	4	3	3	1
HCM Model	95th Queue Length	6	2	3	1
	Error, Veh	2	1	0	0
Simulation Model	95th Queue Length	5	3	3	1
	Error, Veh	1	0	0	0

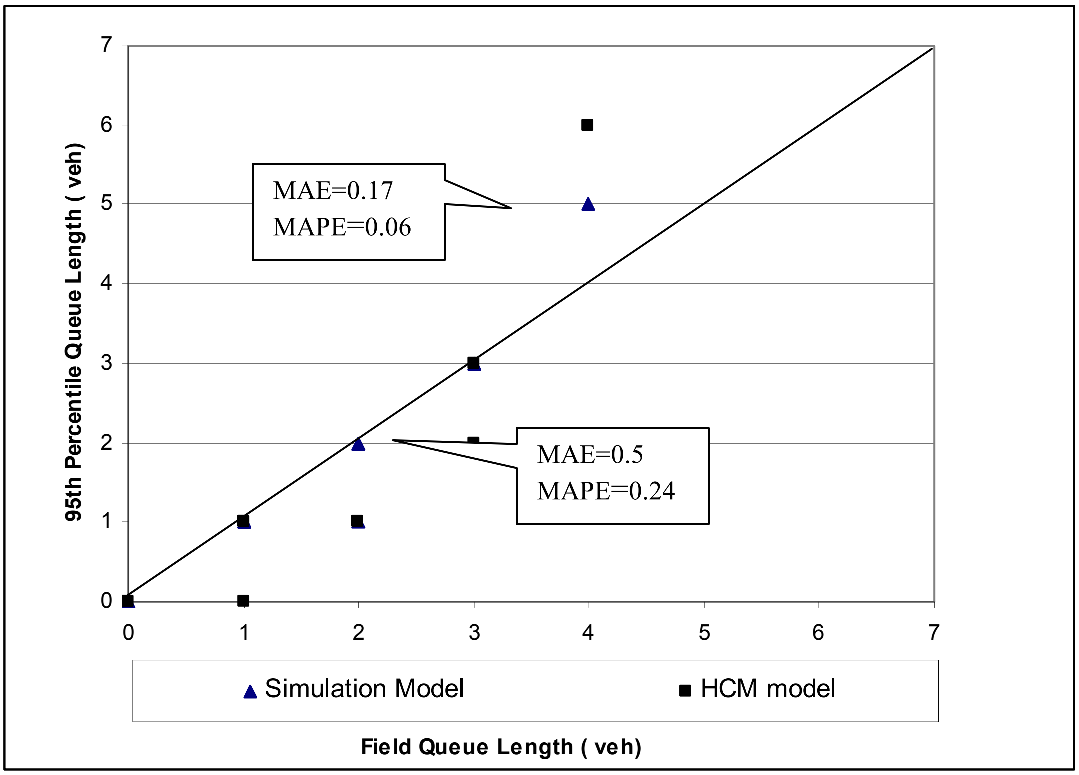


Figure 5.36 The 95th Percentile Queue Length Comparisons Between the HCM 2000 model and the Simulation Model

5.6.3 Service Time Validation

The service time data collected in the field and the predicted service time from the HCM 2000 model and the simulation model are shown in Table 5.11, Table 5.12 and Table 5.13.

Because of obstructions from trees during video taping, the service times for EB at the intersection of 27 St. and Alabama St., and NB and SB at the intersection of Harvard Rd. and Crestline Dr. couldn't be collected from the data professing program. Figure 5.37 shows the comparisons between the service time results predicted by the two models and the field observed service time. In general, the HCM 2000 model predicts longer service times than the simulation model, with the average estimate error of 0.58 sec. This may be caused by the shorter move-up time, which was discussed earlier in this Chapter. Compared to the HCM 2000 model, the simulation model predicts better results, with a MAE value of 0.27 sec. and a MAPE value of 0.09. As shown in Table 5.6, the service times predicted by the simulation model are within 33.33% difference (0.6 sec) of the field observed service times, while 38.89% (1.0 sec) for the HCM 2000 model.

Table 5.11 Service Time Comparisons Between the Field Observation and the HCM 2000 Model and Between the Field Observations and the Simulation Model (27 St. and Alabama St.)

27St. and Alabama St.		NB	WB	SB	EB
FIELD	Service Time	3.1	1.8	2.8	N/A
HCM Model	Service Time	3	2.5	2.4	2.5
	Error, %	-3.23	38.89	-14.29	N/A
Simulation Model	Service Time	3.00	2.40	2.60	2.50
	Error, %	-3.23	33.33	-7.16	N/A

Table 5.12 Service Time Comparisons Between the Field Observation and the HCM 2000 Model and Between the Field Observations and the Simulation Model (Harvard Rd. and Crestline Dr.)

Harvard Rd. and Crestline Dr.		EB	NB	WB	SB
FIELD	Service Time	2.5	N/A	2.7	N/A
HCM Model	Service Time	2.5	2.2	2.8	2.4
	Error, %	0.00	N/A	3.70	N/A
Simulation Model	Service Time	2.5	2.7	3.2	2.4
	Error, %	0.00	N/A	18.52	N/A

Table 5.13 Service Time Comparisons Between the Field Observation and the HCM 2000 Model and Between the Field Observations and the Simulation Model (87 St. and Lamar Ave.)

87 St. and lamar Ave.		SB	EB	NB	WB
FIELD	Service Time	2.8	3.8	3.1	4.1
HCM Model	Service Time	3.8	4.4	4.1	4.9
	Error, %	35.71	15.79	32.26	19.51
Simulation Model	Service Time	2.9	3.4	3.0	3.6
	Error, %	2.04	-10.65	-3.35	-13.13

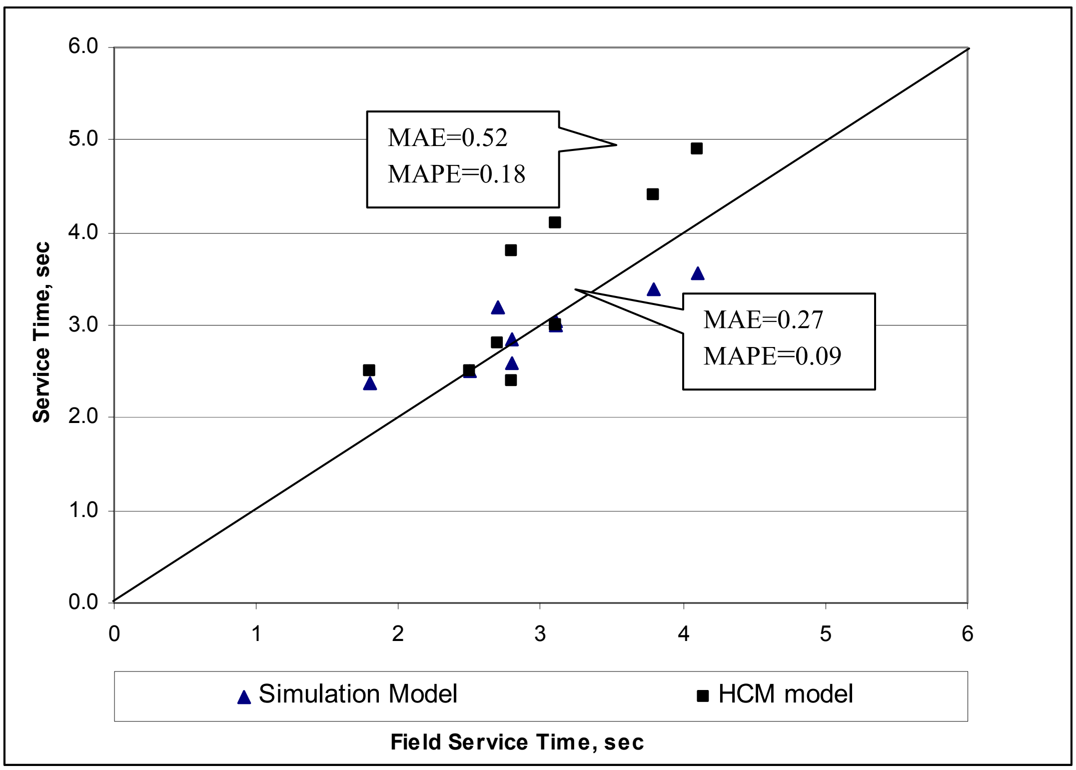


Figure 5.37 Service Time Comparisons Between the HCM 2000 Model and the Field Data, and Between the Simulation Model and the Field Data

5.6.4 The 95th Percentile Queue Length Comparisons Between Different Models

Besides the HCM 2000 model, several popular modern computer programs, such as aaSIDRA, Synchro/Simtraffic, and TSIS, are used by professionals in the transportation industry. In this section, the queue lengths were compared between the simulation model, the HCM 2000 model, aaSIDRA, Synchro / SimTraffic, and TSIS, based on the field data collected in this study. The field data for the 95th percentile queue lengths from 36 approaches at nine intersections (eight intersections in Lawrence and one in Overland Park) will be used in the following discussion.

Figure 5.38 presents the comparisons between the simulation queue lengths and the field observations. The comparisons between the simulation model and the HCM 2000 model are shown in Figure 5.39.

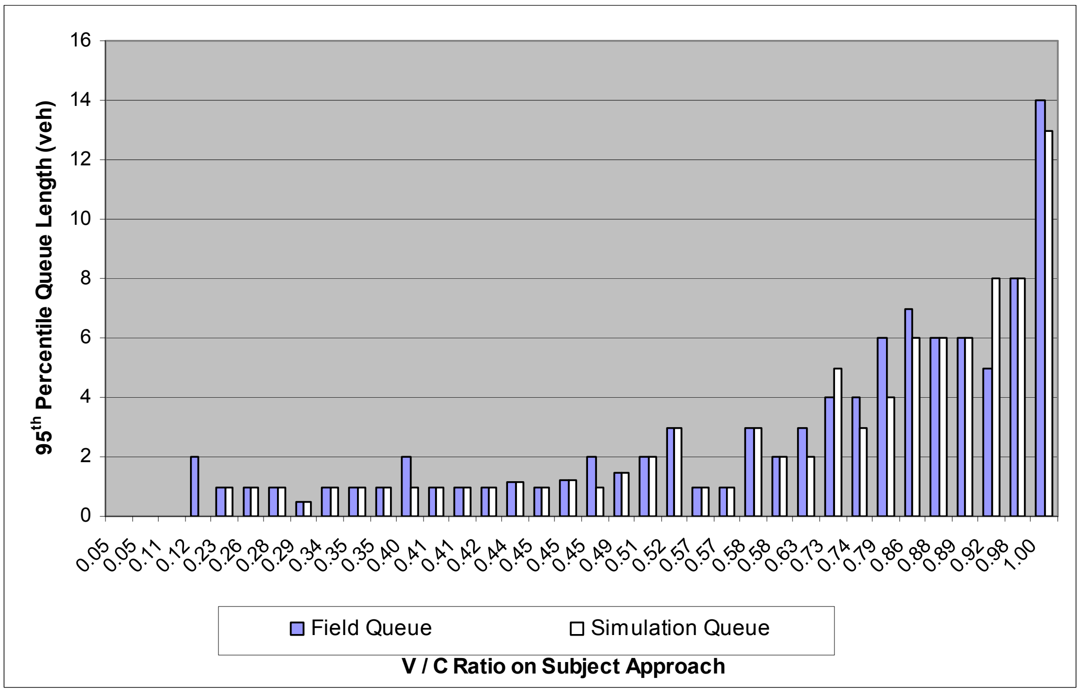


Figure 5.38 The 95th Percentile Queue Length Comparison Between the Simulation Model and the Field Data

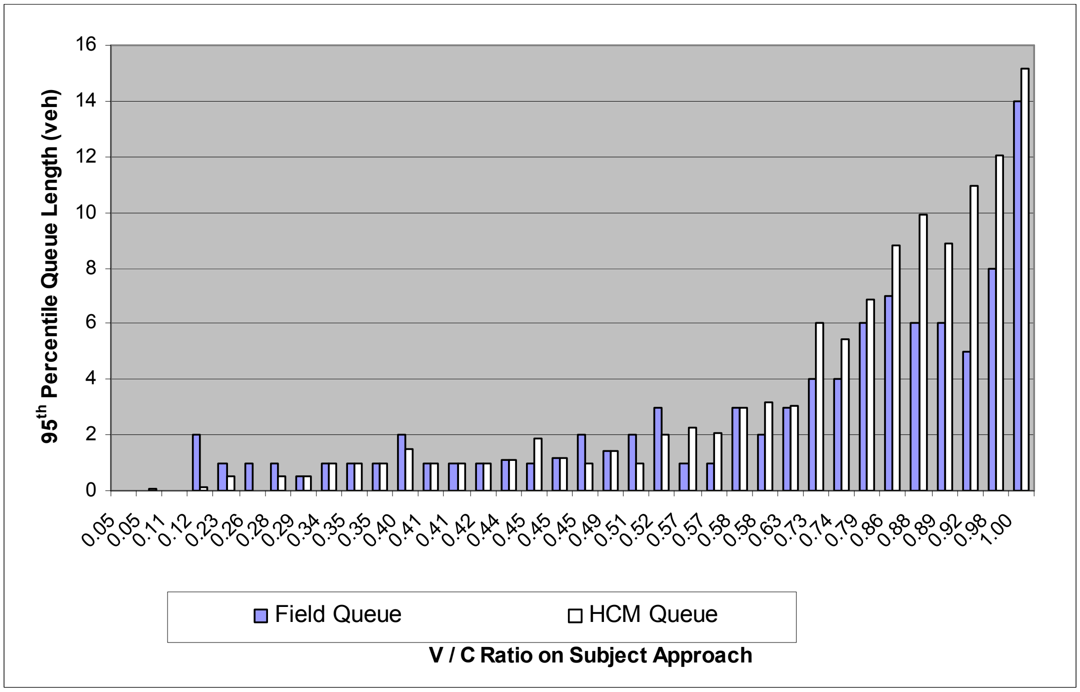


Figure 5.39 The 95th Percentile Queue Length Comparison Between the HCM 2000 Model and the Field Data

The SIDRA (Signalized and Unsignalized Intersection Design and Research Aid) developed by the Australian Road Research Board (ARRB) and Akcelik Associates is used for the design and evaluation of single intersections, including signalized and unsignalized intersections and roundabouts. It is recognized by the US Highway Capacity Manual and FHWA Roundabout Guide and is extensively used in the USA. SIDRA 4.0 was used to calculate queue lengths in this study. The 95th percentile queue length is one of its outputs. The calculation results for the 95th percentile queue lengths are shown in Figure 5.40.

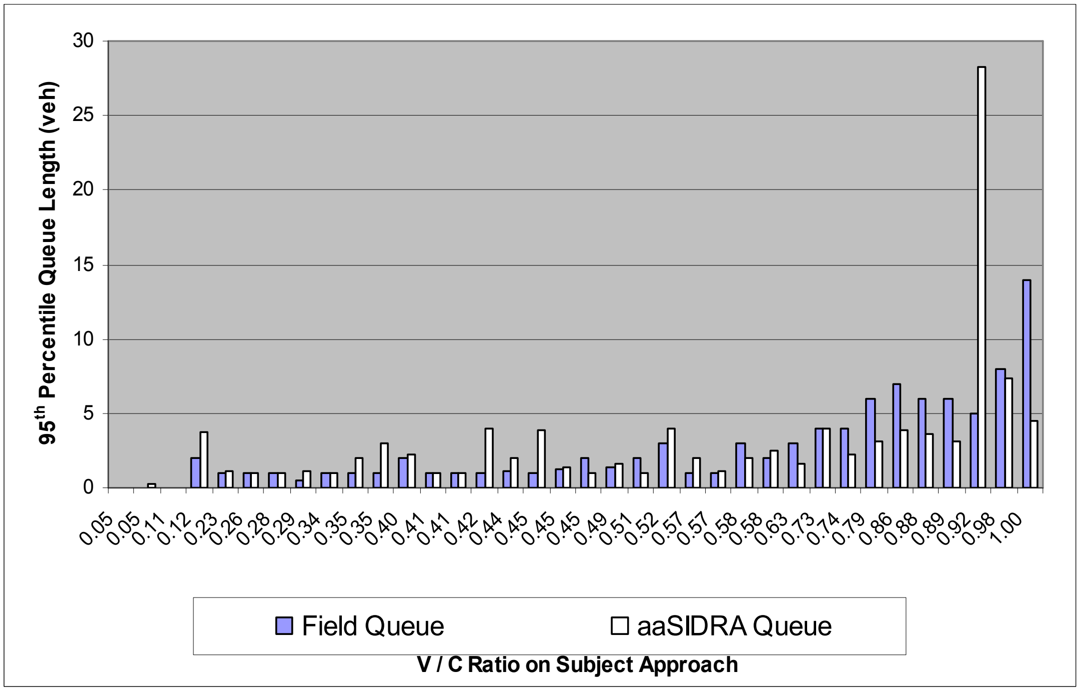


Figure 5.40 The 95th Percentile Queue Length Comparison Between the aaSIDRA Program and the Field Data

Synchro/ SimTraffic was developed by Trafficware and is one of the most used signal timing and simulation software by industries. Synchro is a macroscopic capacity analysis and optimization model. SimTraffic is a more realistic microscopic simulation model and it can fully simulate signalized and unsignalized intersections. Synchro / SimTraffic 6.0 was used in the analysis. It reports the 95th percentile queue length as an output parameter. The simulation results for the 95th percentile queue lengths are shown in Figure 5.41.

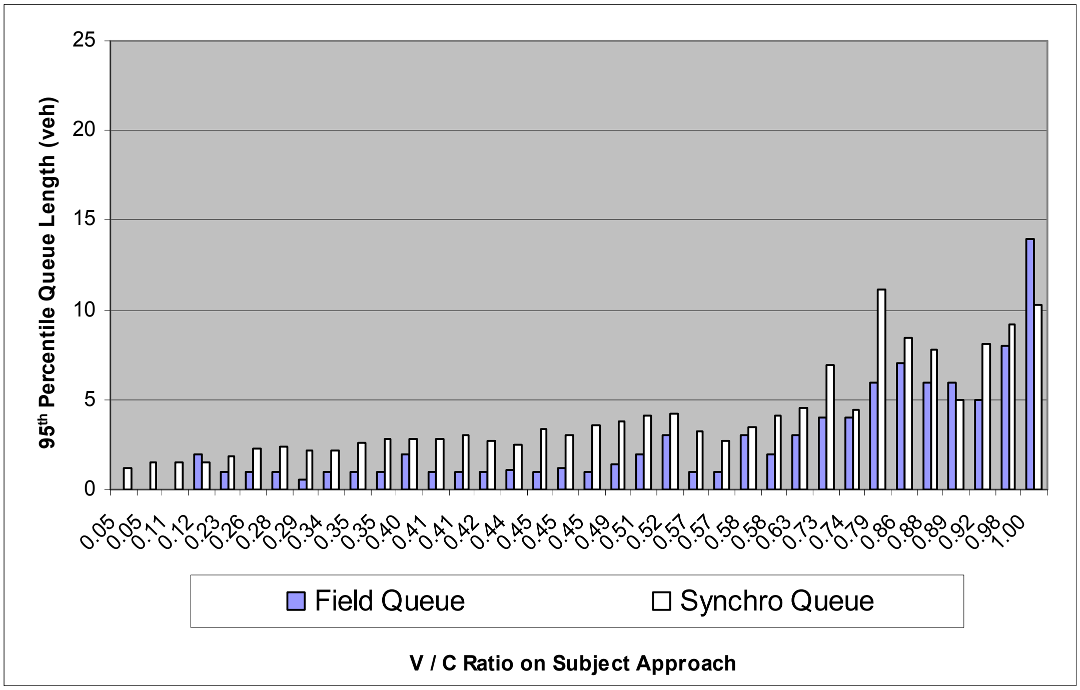


Figure 5.41 The 95th Percentile Queue Length Comparison Between the Synchro Program and the Field Data

CORSIM was developed by the Federal Highway Administration, and is part of the Traffic Software Integrated System (TSIS). It is a comprehensive microscopic traffic simulation, which combines NETSIM for surface streets and FRESIM for freeway streets. CORSIM provides the maximum queue length and average queue length. The average queue consists of the average queue during the entire simulation and is related to the number of seconds that the queues exist. The maximum queue is the longest queue length observed during the simulation. It was reported that the maximum queue length from CORSIM was comparable to the average queue length from Synchro/Simtraffic. Hence, the maximum queue length is used in the comparisons. The maximum queue lengths from CORSIM are shown in Figure 5.42.

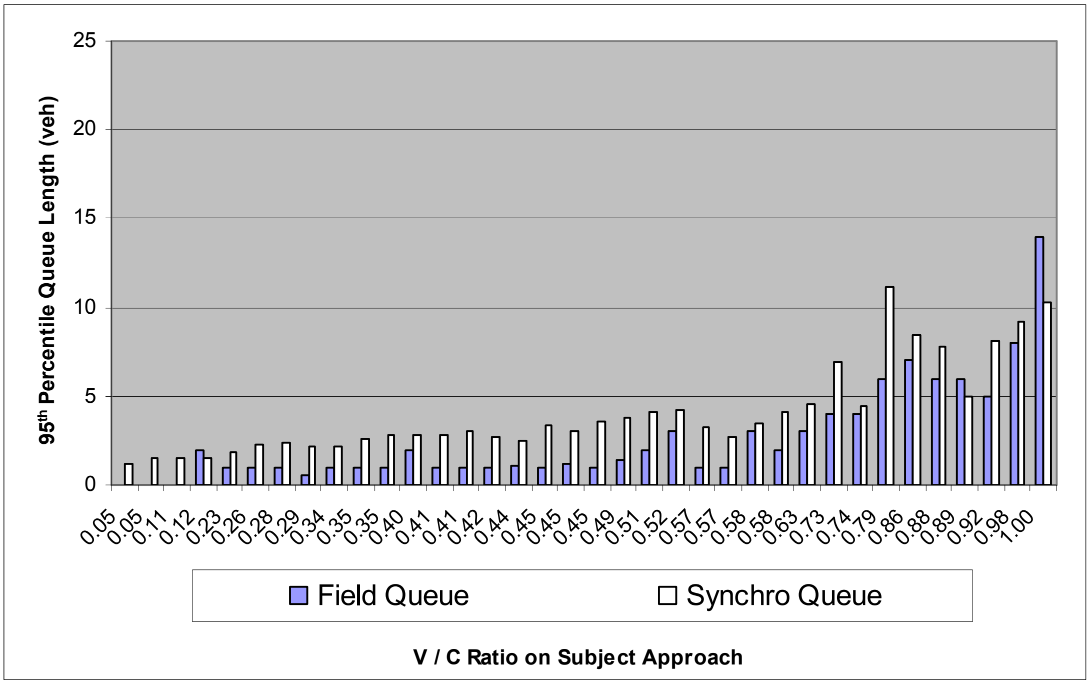


Figure 5.42 The 95th Percentile Queue Length Comparison Between the TSIS Program and the Field Data

The root mean square error (RMSE) is a measure of the differences between queue lengths predicted by a model and the queue lengths actually observed in the field. It was used as an additional evaluation parameter along with mean absolute error (MAE) and mean absolute percent error (MAPE). The mean square error and the root mean square error are defined in Equations (5.7) and (5.8), respectively.

$$\mathrm{MSE} = \frac{1}{n}\sum_{i=1}^{n}(q_{mi} - q_{fi})^2 \tag{5.7}$$

$$\mathrm{RMSE} = \frac{1}{\sqrt{n}}\sqrt{\sum_{i=1}^{n}(q_{mi} - q_{fi})^2} \tag{5.8}$$

Where,

q_{mi} = queue length of data sample from a model

q_{fi} = queue length of data sample from field observations, and

n = total number of data samples

The 95th percentile queue length predicted by the different models were compared with the field observed queue length. Table 5.7 tabulate the MAE, MAPE, and RMSE values based on the 95th percentile queue length comparisons between the different models and the field data. Figures 5.38 thru 5.42 show the 95th percentile queue length comparisons between different models and field observed data. As noted earlier, the field observed queue lengths come from the nine selected intersections in the current study.

Table 5.14 MAE, MAPE, and RMSE Between the 95th Percentile Queue Lengths Predicted by the Different Models and the Field Data

Model	MAE	MAPE	RMSE
Simulation	0.36	0.09	0.80
HCM	0.99	0.35	1.7
SIDRA	1.84	0.64	4.42
Synchro/SimTraffic	1.76	1.04	1.98
TSIS	1.43	0.33	2.67

Figure 3.38 shows the 95th percentile queue length comparison between the simulation model and the field data. When the volume / capacity (V/C) ratio is less than 0.60, the simulation model predicts the 95th percentile queue lengths that are very close to the field observation. Among the four different prediction approaches, the simulation model appears to be the most reliable prediction model. As shown in Table 5.14, the simulation model exhibits the lowest values of MAE, MAPE, and RMSE, and they are 0.36, 0.09, and 0.80, respectively.

The 95th percentile queue length comparison between the HCM 2000 model and the field data is shown in Figure 5.39. Similar to the simulation model, when the V/C ratio is small, the 95th percentile queue lengths predicted by the HCM 2000 model are close to those observed in the field. When the V/C ratio is greater than 0.5, the 95th percentile queue lengths based on the HCM 2000 model are generally longer than the field data. As shown in Table 5.14, among the four prediction approaches, the HCM 2000 model is only secondary to the simulation model developed in this study, with MAE, MAPE, RMSE values of 0.99, 0.35, and 1.7, respectively.

The 95^{th} percentile queue lengths predicted by the aaSIDRA program were compared with the field data and the results are shown in Figure 5.40. As shown in Figure 5.40, when the V/C ratio is greater than 0.63, the 95^{th} percentile queue lengths calculated by the aaSIDRA program are generally shorter than the field observation data, with the only exception when the V/C ratio is 0.92. Another observation of Figure 5.40 is that there are four locations where the aaSIDRA program significantly overestimate the 95^{th} percentile queue length compared with the field observed queue length data. The V/C ratios at those four locations are 0.12 at Haskell Approach 2 with a right turn percentage of 82%, 0.42 at Harper Approach 3 with a right turn percentage of 56%, 0.45 at Connecticut Approach 3 with a right turn percentage of 47%, and 0.92 at Barker Approach 1 with a right turn percentage of 59%. Apparently, the aaSIDRA program may significantly overestimate the 95^{th} percentile queue length when the right turn volume percentage is high.

Figure 5.41 shows the 95^{th} percentile queue length comparison between the results calculated by the Synchro program and the field data. As shown in Figure 5.41, the Synchro program seems to overestimate the 95^{th} percentile queue lengths under low traffic demands when compared with the field data. However, when the traffic demand reaches its capacity, the program tends to underestimate the queue lengths.

The comparison between the TSIS program and field data is shown in Figure 5.42 for the 95^{th} percentile queue length. When the V/C ratio is small, the 95^{th} percentile queue lengths estimated by the TSIS program generally match the field data well. When the V/C ratio is greater than 0.58, the 95^{th} percentile queue lengths predicted by the TSIS program exhibit similar trends to those predicted by the aaSIDRA program. The TSIS program predicts the 95^{th} percentile queue lengths much shorter than the field data.

As shown in Table 5.14, the statistical parameters MAE, MAPE, and RMSE for the aaSIDRA, Synchro, and TSIS programs are significantly higher than those for the simulation model and the HCM 2000 model. Thus the 95th percentile queue lengths results predicted by the aaSIDRA, Synchro, and TSIS programs are much less reliable than those based on the simulation model and the HCM 2000 model.

5.7 CAPACITY FOR AWSC INTERSECTIONS

In the HCM Manual, capacity is defined as “the maximum throughput on an approach given the flow rates on the other intersection approaches”. The capacity is reached when the degree of saturation on one approach is approximately equal to 1.0. The corresponding delay was reported usually just above 50 sec/veh. It is very difficult to judge if an intersection reaches its capacity in the queue simulation model. The AWSIM model defines capacity as the maximum flow rate when the average control delay is less than 60 sec/veh. It is a tedious process to run the simulation model to reach capacity at one traffic scenario.

In this research, a capacity simulation model was developed separately using the same theory as used in the queue length simulation model to study the capacity features for FWSC intersections. It was assumed that approach one was saturated, which simply means there is always a car on this approach. Based on the volume splits between the major street and the minor street and the volume directional distributions, vehicles were generated for the other approaches. Then the same procedures, as that used for the queue length simulation model, were used. The capacity at an intersection is the total vehicles processed by the intersection. Figure 5.43 shows the model inputs required for the capacity simulation. The simulation flow chart, as shown in Figure 5.44, illustrates the procedure to calculate capacities at AWSC intersections.

5.7.1 Inputs and Outputs

Figure 5.43 shows the inputs required for the capacity simulation at FWSC intersections. The V (minor)/V (total) parameter is the volume percentage of the minor street to the total volumes of the intersection. For the major street, the volume ratio is denoted as V (major)/V (total). LT, ST and RT are the volume percentages of left turn vehicles, through vehicles, and right turn vehicles. P1 and P3 represent the major street directional volume distributions, while P2 and P4 are the directional distributions for the minor street. The output is the capacity for the intersection according to the given volume splits and directional volume distributions. Twenty runs were made to generate stable results.

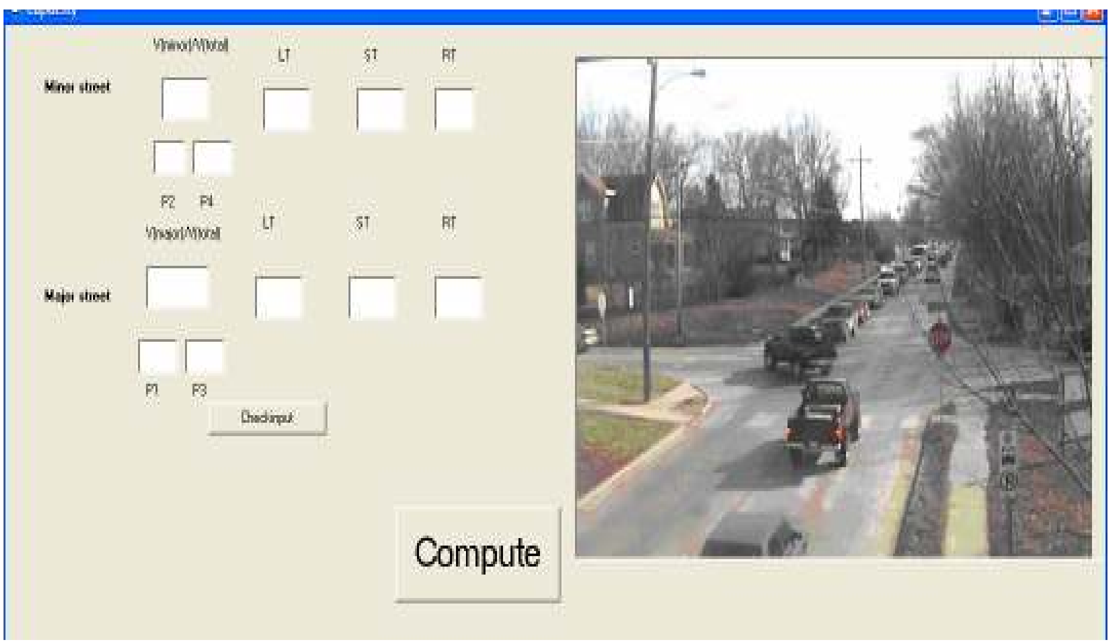
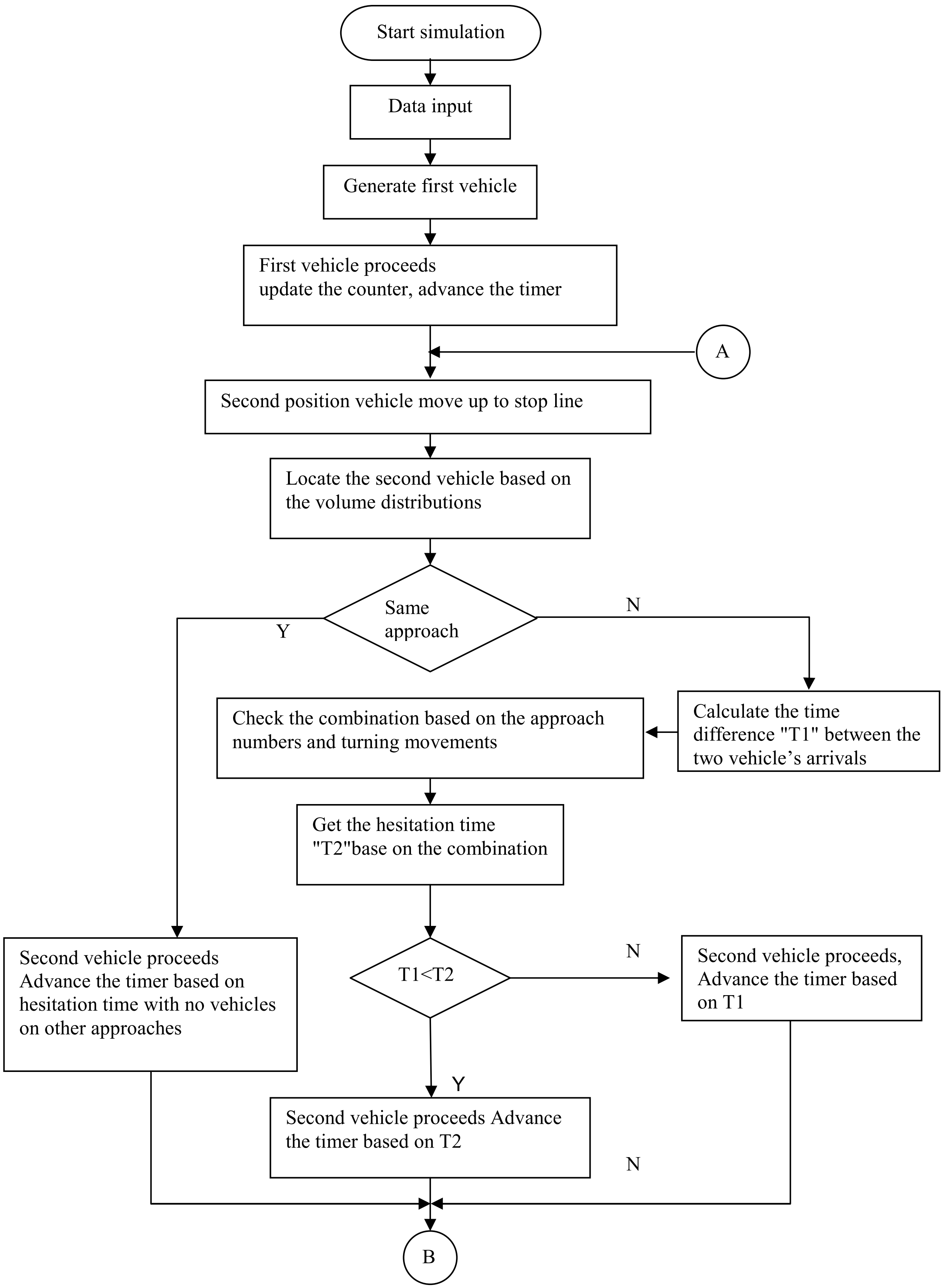


Figure 5.43 Capacity Simulation Model Inputs for AWSC Intersections


Start simulation
Data input
Generate first vehicle
First vehicle proceeds
update the counter, advance the timer
A
Second position vehicle move up to stop line
Locate the second vehicle based on the volume distributions
Same approach
Y
N
Calculate the time difference "T1" between the two vehicle's arrivals
Check the combination based on the approach numbers and turning movements
Get the hesitation time "T2"base on the combination
Second vehicle proceeds Advance the timer based on hesitation time with no vehicles on other approaches
T1<T2
N
Second vehicle proceeds, Advance the timer based on T1
Y
Second vehicle proceeds Advance the timer based on T2
N
B

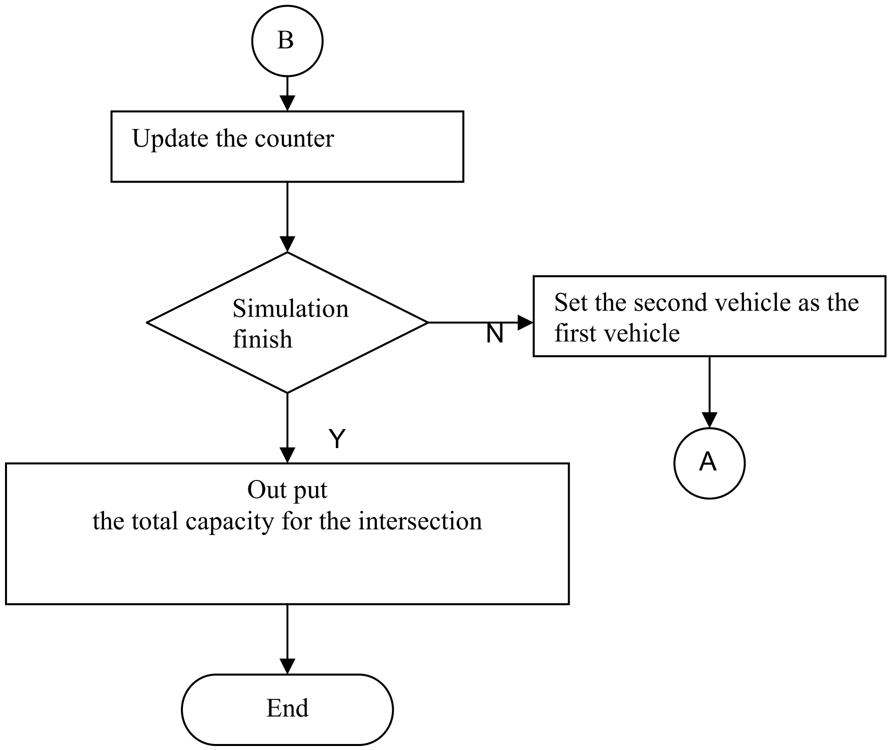


Figure 5.44 Capacity Simulation Flow Charts for AWSC Intersections

5.7.2 Capacity Comparisons Between the Simulation and HCM 2000 Models

The total capacities of FWSC intersections from the simulation model are presented in Table 5.15. The capacity calculation based on the HCM 2000 model are tabulated in Table 5.16. Figure 5.45 and Figure 5.46 shows the simulation results. As shown in Table 5.15, the intersection reaches its highest capacity when there is even traffic demand on the major street and minor street (volume ratio is 50/50 between the minor and major streets). This is consistent with other studies. As shown in Table 5.17, the capacity values predicted by the simulation model are generally higher than those based on the HCM 2000 model, but not significantly higher. The capacity values from the simulation model follow the trend that the capacity decreases first with the increasing minor street traffic demand, and then starts to gain back, which is clearly shown in Figures 5.45 and 5.46.

Table 5.15 Total Intersection Capacity at Different Volume Splits, Based on the Simulation Model

Total Intersection Capacity From Simulation			
Volume Split Minor/Major	L/T/R 0/1/0	L/T/R 0.1/0.8/0.1	L/T/R 0.2/0.6/0.2
0/100	1599	1517	1456
10/90	1584	1492	1449
20/80	1574	1489	1445
30/70	1569	1502	1468
40/60	1577	1515	1494
50/50	1599	1543	1524

Table 5.16 Total Intersection Capacity at Different Volume Splits and Different Turning Combinations, Based on the HCM 2000 Model (Wu, 2002)

Total Intersection Capacity From HCM 2000			
Volume Split Minor/Major	L/T/R 0/1/0	L/T/R 0.1/0.8/0.1	L/T/R 0.2/0.6/0.2
0/100	1549	1536	1523
10/90	1483	1472	1462
20/80	1450	1441	1432
30/70	1441	1433	1425
40/60	1452	1445	1438
50/50	1483	1477	1470

Table 5.17 Intersection Capacity Difference Between the Simulation Model and the HCM 2000 Model, at Different Volume Splits

Total Intersection Capacity Difference			
Volume Split Minor/Major	L/T/R 0/1/0	L/T/R 0.1/0.8/0.1	L/T/R 0.2/0.6/0.2
0/100	+50	-19	-67
10/90	+101	+20	-13
20/80	+124	+48	+13
30/70	+128	+69	+43
40/60	+125	+70	+56
50/50	+116	+66	+54

A similar trend observed for both the simulation model and the HCM 2000 model is that intersection capacities are not very sensitive to the volume splits between the minor and major streets. This may result from the two phase traffic operation pattern. The more traffic volume on the minor street, the more right of way alternative operations occur between the two streets. As a result, the required service time is longer than that for traffic combinations on only one street. On the other hand, more traffic combinations occur on the minor street with increasing volume on the minor street. These two “alternative” and “combination” operations are compensative to each other. At first the “alternative” operations are more significant than “combination” operations, so the capacity decreases. With increasing volume on the minor street, more “combination” operations occur and the capacity begins to increase after the “breaking points”, where the capacity starts to increase from the lowest point. For the simulation model, the “breaking points” are at the volume ratio of 20/80, except in the first scenario with through vehicles only, which has a “breaking point” at a volume ratio of 30/70. At the same time, the “breaking points” are at the volume ratios of 30/70 for the HCM 2000 model.

A major difference was observed between the HCM 2000 model and the simulation model. For the HCM 2000 model, the capacity at a volume split of 50/50 is less than that at volume split of 0/100. However, such a trend was not observed for the simulation model. As mentioned earlier, the intersection reaches its highest capacity at a volume split of 50/50 based on the simulation model. The reason for this variance may be that the HCM 2000 model overestimates the conflicts between the approaches when "alternative" operations occur. Also, the 2-second move-up time used in the HCM 2000 model may contribute to this, too, because it attributes to a longer service time.

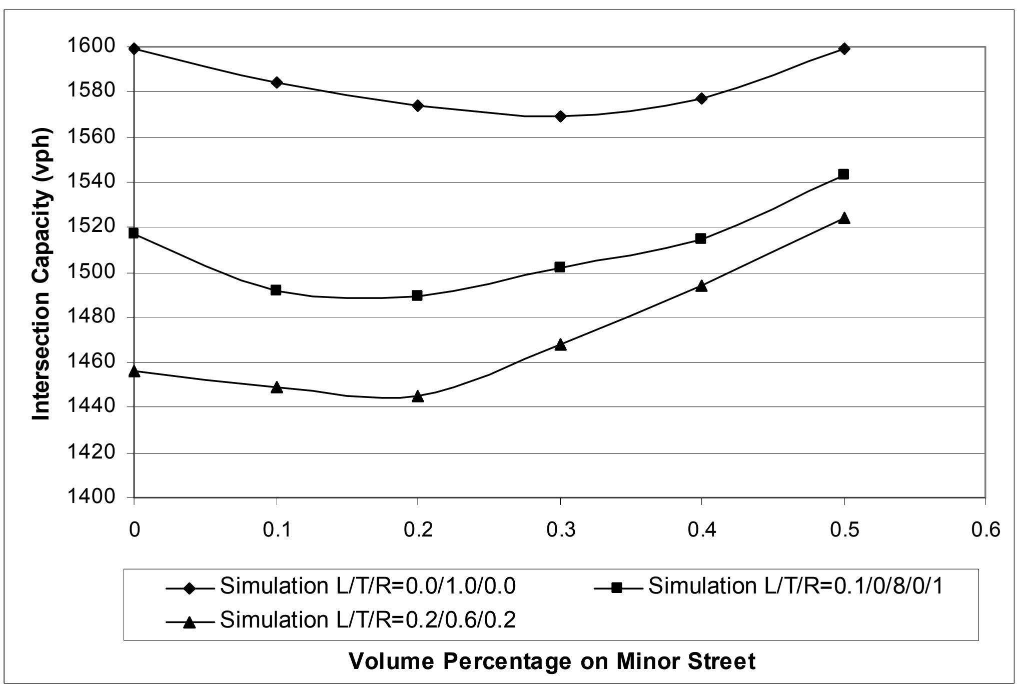


Figure 5.45 Total Intersection Capacities at Different Volume Splits and Different Turning Combinations, Based on the Simulation Model

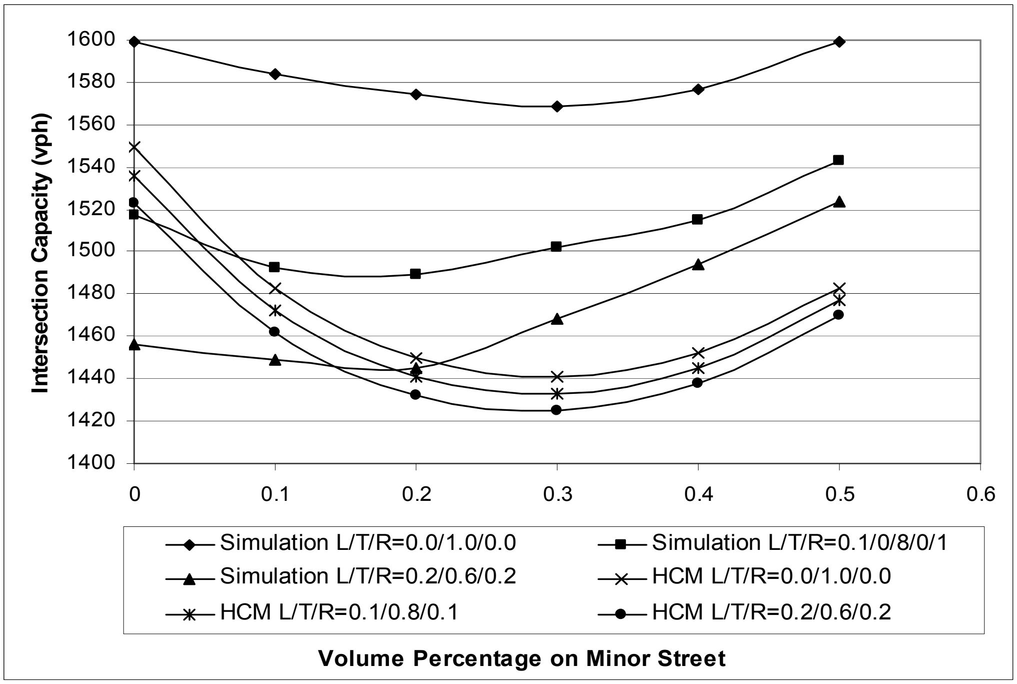


Figure 5.46 Capacity Comparisons Between the HCM 2000 Model and the Simulation Model, at Different Volume Splits and Different Turning Combinations

5.7.3 Capacity Comparisons between the Simulation and Wu's ACF Models

Wu developed the Addition-Conflict-Flows (ACF) model for calculating capacities for AWSC intersections, which was derived from graph theory. It incorporated the HCM 2000 approach, based on five conflict cases and expanded it to 192 steam-based cases. The modified ACF model used a very complicated iterative process.

The total capacities of an intersection at different volume splits calculated from the modified ACF model are tabulated in Table 5.18. Figure 5.47 shows the comparisons between the simulation model and the modified ACF model. The capacity values based on the modified ACF model are significantly higher than those predicted by the simulation model, as shown in Tables 5.18 and 5.19. This indicates that the modified ACF model might underestimate the degree of conflicts, such as the cases involving the subject approach right turn vehicles. Theoretically, the right turn vehicles can make right turns without waiting for their turn. And thus, they probably need very short service time. However, based on the collected data, the service time for right turn vehicles is only about 17% shorter than that for through vehicles. And the service time for left turn vehicles is about 20% longer than that for through vehicles.

Table 5.18 Total Capacity at Different Volume Splits and Different Turning Combinations, Based on the Wu's Modified ACF Model

Total Intersection Capacity From Wu's ACF Model			
Volume Split Minor/Major	LT/TH/RT 0/1/0	LT/TH/RT 0.1/0.8/0.1	LT/TH/RT 0.2/0.6/0.2
0/100	1846	1792	1760
10/90	1765	1747	1750
20/80	1758	1752	1773
30/70	1803	1796	1826
40/60	1897	1881	1912
50/50	2057	2015	2041

Table 5.19 Intersection Capacity Difference Between the Simulation Model and the ACF Model, at Different Volume Splits

Total Intersection Capacity Difference			
Volume Dplit Minor/Major	L/T/R 0/1/0	L/T/R 0.1/0.8/0.1	L/T/R 0.2/0.6/0.2
0/100	-247	-275	-304
10/90	-181	-255	-301
20/80	-184	-263	-328
30/70	-234	-294	-358
40/60	-320	-366	-418
50/50	-458	-472	-517

As shown in Figure 5.47, FWSC intersections reach the highest capacity at the volume split of 50/50 based on the ACF model. Another observation can be made that the capacity at the volume split of 50/50 is significantly higher than that at the volume split of 0/100.

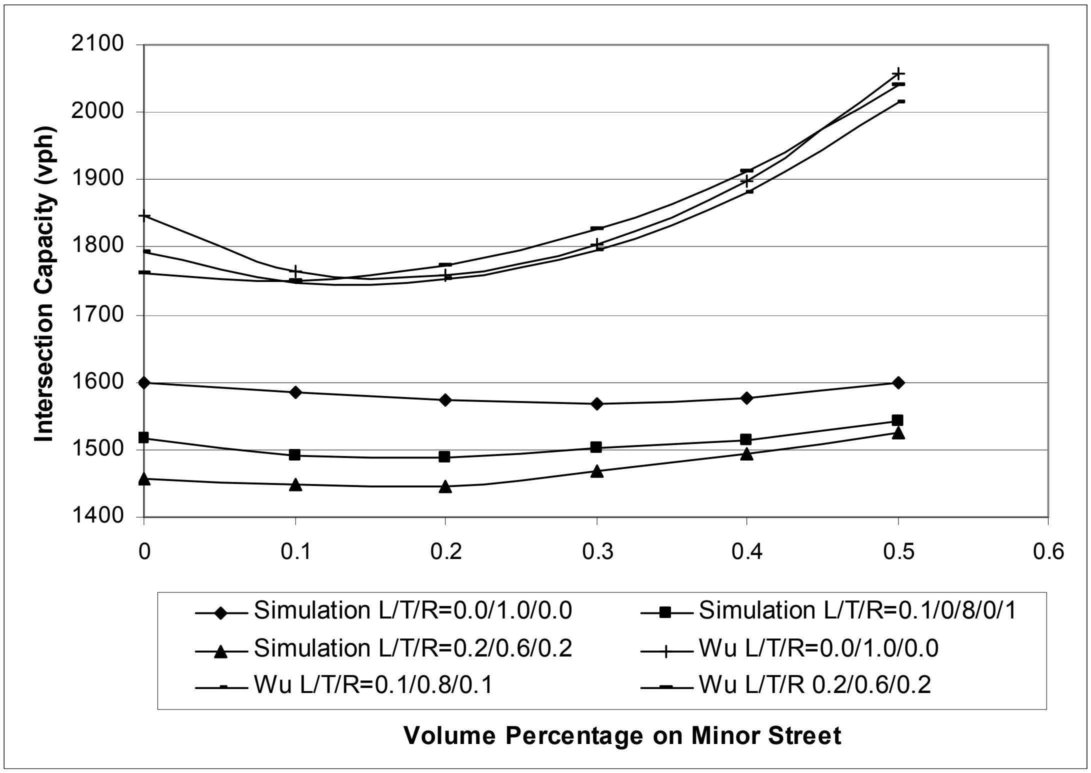


Figure 5.47 Capacity Comparisons Between Wu's ACF Model and the Simulation Model, at Different Volume Splits and Different Turning Combinations

5.7.4 Capacity Based on Other Models

Table 5.20 shows the total capacities at different volume splits from other studies. The comparisons between these models and the simulation model are shown in Figure 5.48. Similar to Wu's ACF mode, the capacity estimations from these models are generally higher than those based on the simulation model in this study, with the exception of the AWSIM model.

Table 5.20 Total Capacity at Different Volume Splits Based on the Other Models (Tian et al, 2001)

Total Intersection Capacity From Other Different Models				
Volume Split Minor/Major	LT/TH/RT 0/1/0			LT/TH/RT 0.2/0.6/0.2
	Hebert	Richardson	AWSIM	AWSIM
0/100	1800	1800	1650	1400
10/90				
20/80				
30/70				
40/60	1700	1650	1900	1600
50/50	1900	1900	2000	1650

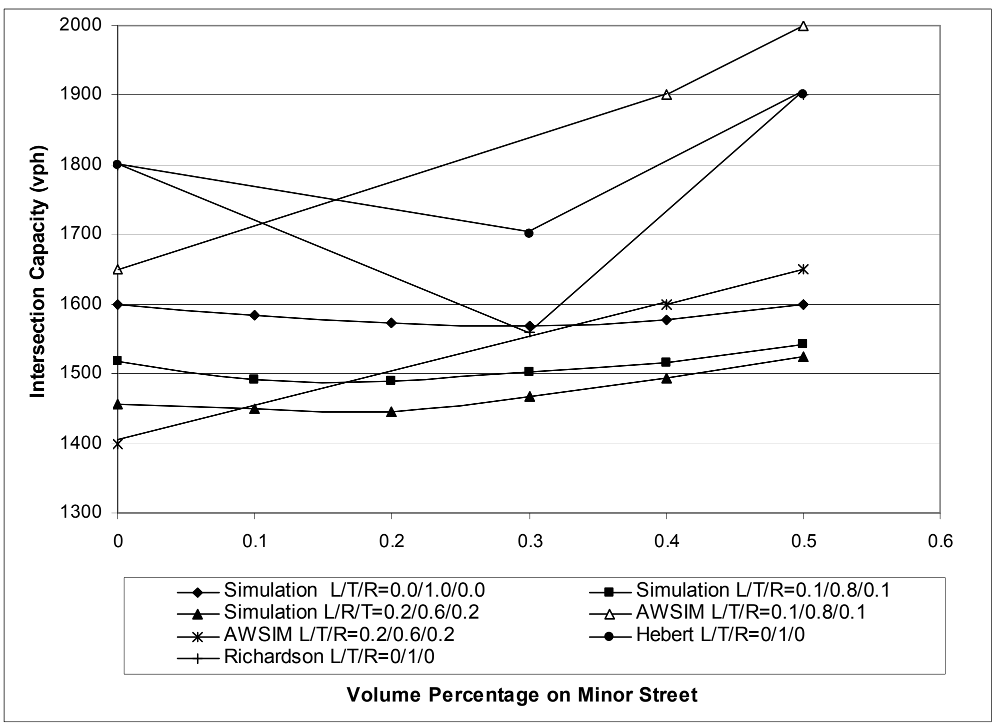


Figure 5.48 Capacity Comparisons between Other Models and the Simulation Model, at Different Volume Splits and Different Turning Combinations

5.7.5 Effects of Volume Directional Distribution on Intersection Capacity

Simulation runs were conducted to study the effect of the volume directional distributions. The following assumptions were made: no traffic demand on the minor street and the directional distributions ranged from 0 to 0.5. Figure 5.49 shows that the intersection capacity increases exponentially when directional distribution on the major street increases. Both the HCM 2000 model and the simulation model demonstrate very similar relationships between intersection capacities and directional distributions on the major street. But the simulation model is slightly more sensitive to the volume directional distribution than the HCM 2000 model.

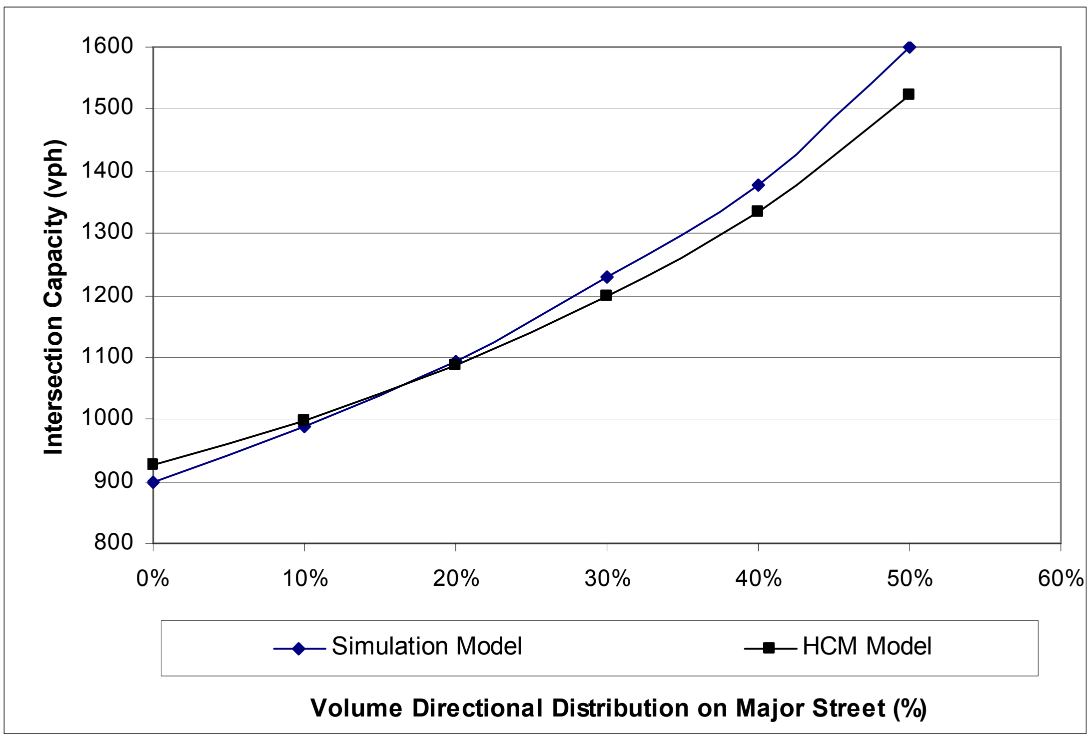


Figure 5.49 Effects of Volume Directional Distribution on Intersection Capacities, Based on the Simulation Model and the HCM 2000 Model

5.7.6 Effects of Right and Left Turn Traffic on Intersection Capacity

Effects of right turn and left traffic on the capacities were also investigated based on the simulation model. Figure 5.50 and Figure 5.51 show the comparisons between the two models. The HCM 2000 model shows nearly constant capacity under different right turn or left turn vehicle percentages. However, for the simulation model, Figure 5.50 shows that intersection capacities linearly increase as the percentage of right turn vehicles increases. Accordingly, as the percentage of left turn vehicles increases, intersection capacities drop linearly as shown in Figure 5.51.

Since right turn vehicles need shorter service time than through vehicles, it is reasonable that a higher percentage of right turn traffic can increase the intersection capacity. On the other hand, left turn traffic needs longer service time than through vehicles, which has a negative impact on the intersection capacity. Therefore, it is not reasonable for the HCM 2000 model to have a nearly constant capacity with increasing right or left turn traffic. Figures 5.50 and 5.51 indicate that the HCM 2000 model does not have the ability to consider the effects of different percentages of right / left turn traffic. Thus, the HCM 2000 model cannot reflect the impacts of right/left turn movements on FWSC intersection capacities.

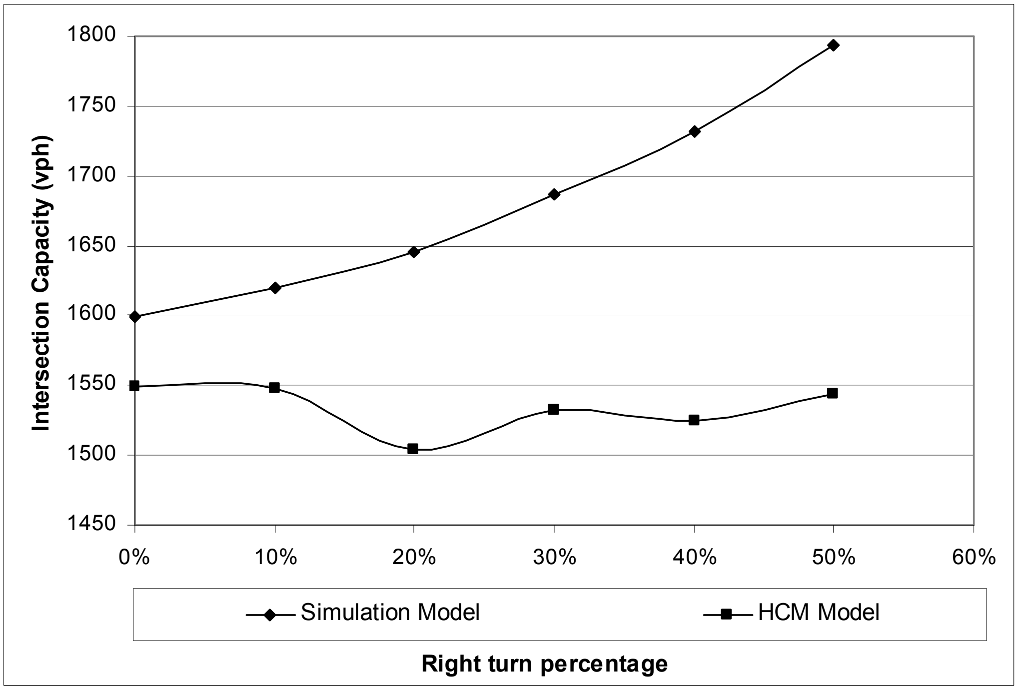


Figure 5.50 Effects of Right Turn Traffic on Intersection Capacities, Based on the Simulation Model and the HCM 2000 Model

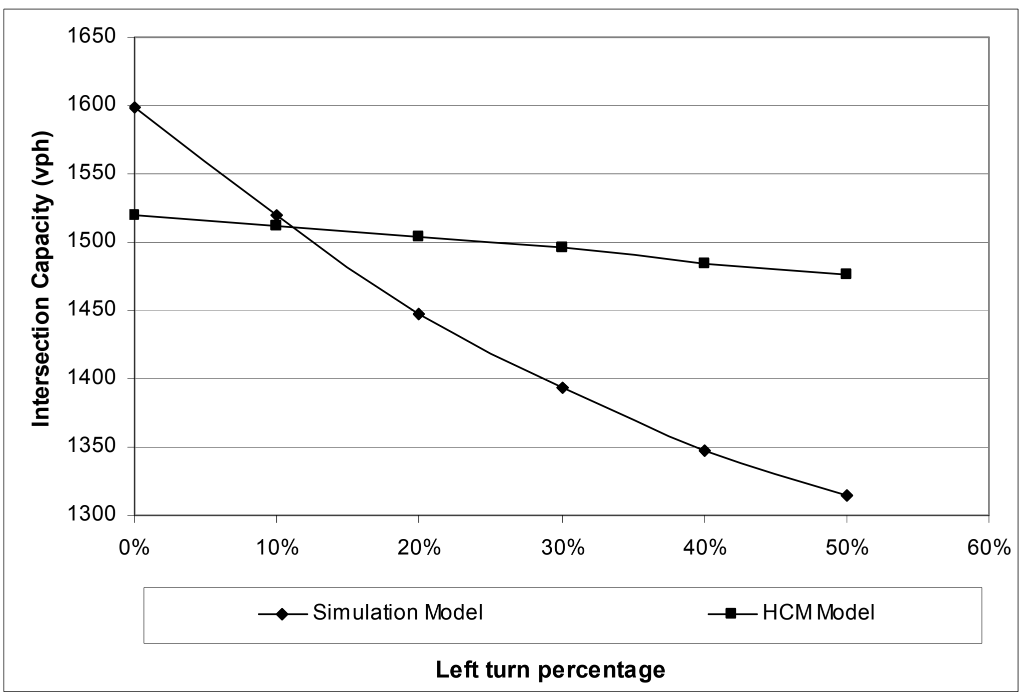


Figure 5.51 Effects of Left Turn Traffic on Intersection Capacities, Based on the Simulation Model and the HCM 2000 Model

5.8 CONCLUSIONS AND DISCUSSIONS

Based on the hesitation time for 27 turning movement combinations, simulation models were developed to evaluate the 95th percentile queue lengths, average control delays and average service time. The simulation results were based on the field data collected at six FWSC intersections in Lawrence and validated with the field observations from other three intersections (two intersections in Lawrence, KS and one intersection in Overland Park, KS). It suggested that the simulation models could provide very reasonable predictions for queue lengths, control delays and service time. At the same time, the HCM 2000 model was compared with the simulation model and tested against the field data. It was found that when the volume capacity ratio (v/c) is less than 0.5, the queue lengths from the HCM 2000 model are very close to the field observations. However, when the v/c is greater than 0.5 and less than 1, the HCM 2000 model tends to overestimate the queue lengths. But when traffic demands are oversaturated, it may underestimate the queue lengths.

The HCM 2000 model also predicts longer service time for all the approaches at the seven intersections. This may be due to the short move-up time value of 2 seconds, which is used in the HCM 2000 model. It also showed that the HCM 2000 model generally predicts longer control delays compared to the field data collected at two intersections.

Two queuing models (Tian model and TWSC model) were evaluated in this study. With the control delay values from the simulation model, the Tian model predicted better-fit results to the simulation queue lengths than TWSC model did. However, the TWSC model worked better than Tian model when predicting queue lengths based on the control delay from the HCM 2000 model output. But when the traffic demand is overcapacity, the TWSC model likely underestimates the queue lengths. However, there are not enough field data to support this hypothesis.

The simulation model, and the HCM 2000, TSIS, Synchro and aaSIDRA models were compared against the queue lengths from field observations. It was found that the simulation model could produce the most accurate results compared to the other programs. Next to the simulation model, the HCM 2000 predicts reasonable queue lengths. Synchro tends to predict longer queues under low traffic demands and shorter queues under high traffic demands. Following Synchro, the TSIS model predicts very reasonable queues under low traffic demands but shorter queues under high traffic demands. Among these programs, the aaSIDRA 's results fit the field data the least because it significantly overestimates the queue lengths when right turn percentages are substantial and it underestimates queue lengths under high traffic demands.

After testing the simulation model's validity, the capacity simulation model was developed using the same hesitation time values to calculate the capacities for FWSC intersections under different traffic scenarios. Then several factors were investigated to determine their impacts on intersection capacities.

Capacity values at FWSC intersections based on several studies were compared. It was found that the capacity values predicted by the simulation model were generally higher than the HCM 2000 capacity values under different traffic scenarios. However, the results from the two models were comparable. However the simulation capacities were lower than the predictions from other studies including Wu's ACF model, AWSIM model, Richardson model and Hebert empirical model.

It holds true that an intersection has the best performance under evenly distributed traffic on the two intersected streets with even directional traffic distributions.

Right turn and left turn movements have impacts on the intersection capacity. It is suggested that an intersection could have higher capacity with more right turn volumes. In contrast with right turn movements, more left turn movements could decrease the intersection capacity.

CHAPTER 6 CONCLUSIONS

This study focus on capacity and other traffic analyses for FWSC intersections based on 27 turning movement combinations instead of an approach based technique. Six intersections in Lawrence, Kansas were selected for data collection based on geometric configuration, location and traffic conditions. A video data processing program was developed to extract the needed data to an excel spreadsheet from the videotapes. After the data reductions, the headway distribution, move-up time, acceleration time and deceleration time, reaction time, and hesitation time for the 27 turning movement combinations were analyzed. With the analysis results, a simulation program was developed to evaluate the queue length, control delay and service time. After calibration and validation using collected data in Lawrence and Overland Park, KS for this study, the program proved to be very reliable. For comparative purposes, each scenario was also studied using the HCM 2000 model. At the same time, two queue length models were evaluated, comparing the simulation results and the HCM 2000 model against the field data. Also, arrival headway distribution impacts were analyzed. The features of control delay, queue length and service time were analyzed under different volumes and volume ratios between the major street and the minor Street.

Then the capacity was calculate using simulation based on the calibrated data. With different volume ratios on the two streets and different volume directional distributions, simulations were conducted to predict intersection capacities. Additionally, left turn and right turn traffic impacts on capacity were investigated.

6.1 CONCLUSIONS

The following conclusions can be drawn:

a. The FWSC intersection has higher capacity when the traffic demand is evenly distributed on the two streets with even directional distribution.

b. The capacity from this study is higher than the values calculated from the HCM 2000 model, but it is lower than several other studies. For engineering practice purpose, the HCM 2000 model results can be considered reasonable.

c. Left turn traffic has a negative impact on the intersection capacity and right turn traffic can increase the capacity

d. Longer than 2 sec. move-up time should be used for estimating intersection capacity. It was proved by simulation that short move-up times may cause longer service times compared to the field data.

e. The simulation model was tested against the field data collected at FWSC intersections in Lawrence and Overland Park and it proved to be a very reliable model that can predict the queue length, control delay and service time, which resemble the field data very closely. Further application in VISSIM and other commercial simulation software can be considered.

f. The HCM 2000 model generally predicts longer queue lengths, longer control delays and longer service time.

g. Tian's 95th percentile queue length model, based on the average queue length from Little's formula, predicts better results than the TWSC queue model based on the simulation data. The TWSC model predicts shorter queue length under oversaturated traffic conditions.

h. Arrival headway distributions have impacts on the queue length, control delay and service time. Platoon arrival can increase the queue length, control delay and service time. This is consistent with other studies.

6.2 RECOMMENDATIONS

The simulation model needs significant validation efforts based on massive field data because it was calibrated only with the data collected at six intersections in Lawrence and tested by the data collected at three intersections (two intersections in Lawrence, KS and one intersection in Overland park, KS). Further research is needed to investigate the impacts of speed, speed distributions, flared approaches, two-lane approaches, profile grades and driver behaviors in different regions. Future efforts also should include the consideration of driveways, parking on the streets and pedestrians. The driver behavior differences between peak hour and non-peak hour, rural and urban also need to be investigated.

APPENDIX A
VISUAL BASIC CODE FOR QUEUING MODEL

```
 'define the excel worksheets

Dim xlsheet1 As Excel.Worksheet
Dim xlsheet2 As Excel.Worksheet
Dim xlsheet3 As Excel.Worksheet
Dim xlsheet4 As Excel.Worksheet
Dim xlsheet5 As Excel.Worksheet
Dim xlsheet6 As Excel.Worksheet
Dim xlapp1 As Excel.Application
Dim xlbook1 As Excel.Workbook

Dim comb(1 To 108, 1 To 3) As Single ' for turning movement combinations
Dim realar1(1 To 1000)   ' for arrival vehicles at approach 1
Dim realar2(1 To 1000)   ' for arrival vehicles at approach 2
Dim realar3(1 To 1000)   ' for arrival vehicles at approach 3
Dim realar4(1 To 1000)   ' for arrival vehicles at approach 4

Private Sub Form_Load() ' define the inputs

sl.Text = " "
st.Text = " "
sr.Text = " "
  nl.Text = " "
nt.Text = " "
 nr.Text = " "
el.Text = " "
et.Text = " "
er.Text = " "
wl.Text = " "
wt.Text = " "
wr.Text = " "
sv.Text = " "
nv.Text = " "
wv.Text = " "
ev.Text = " "

Set xlapp1 = CreateObject("excel.application") ' read hesitation time based on turning movement
combinations
 xlapp1.Visible = True
 Set xlbook1 = xlapp1.Workbooks.Open("C:\Documents and Settings\jyin\My
Documents\VB\arrival.xls")
 Set xlsheet1 = xlbook1.Worksheets(1)
 Set xlsheet2 = xlbook1.Worksheets(2)
```

```
 Set xlsheet3 = xlbook1.Worksheets(3)
 Set xlsheet4 = xlbook1.Worksheets(4)
 Set xlsheet5 = xlbook1.Worksheets(5)
 Set xlsheet6 = xlbook1.Worksheets(6)
 Set ep1 = GetObject(, "excel.application")
 xlsheet1.Activate
 xlsheet2.Activate
 xlsheet3.Activate
 xlsheet4.Activate
 xlsheet6.Activate
 Dim k, j As Integer
 For k = 1 To 108
   For j = 1 To 3
  comb(k, j) = xlsheet5.Cells(k, j)
 Next j
 Next k
End Sub

Public Sub ARRIVAL1(tv) ' generate vehicles for approach 1 based on the its volume
 xlsheet1.Activate
 xlsheet1.Cells.Select
   With Selection
      xlsheet1.Cells.ClearContents
   End With
Dim R As Double, t, AT, q, B, V As Double
Dim oo As Single
t = 0
ARRIVETIME = 0
B = 0.6
V = 2
q = Exp(-1 * B * V * tv / 3600)
AT = ((3600 / tv) - V) / q
For i = 1 To tv
Randomize
R = Rnd
Do While R = 1
Randomize
R = Rnd
 Loop

If R <= 1 - q Then
 t = V
 Else
 t = -(Log(1 - R)) * AT + Log(q) * AT + V
 oo = t
 End If
```

```
 ARRIVETIME = ARRIVETIME + t
xlsheet1.Cells(i, 2) = q / AT * Exp(-1 / AT * (oo - V))
xlsheet1.Cells(i, 1) = t
xlsheet1.Cells(i, 3) = ARRIVETIME
realar1(i) = ARRIVETIME
 Next
End Sub

Public Sub ARRIVAL2(tv) ' generate vehicles for approach 2 based on the its volume
 xlsheet2.Activate
  xlsheet2.Cells.Select
    With Selection
        xlsheet2.Cells.ClearContents
    End With
Dim R As Double, t, AT, q, B, V As Double
Dim oo As Single
t = 0
ARRIVETIME = 0
B = 0.6
V = 2
q = Exp(-1 * B * V * tv / 3600)
AT = ((3600 / tv) - V) / q
For i = 1 To tv
Randomize
R = Rnd
Do While R = 1
Randomize
R = Rnd
 Loop
 If R <= 1 - q Then
 t = V
 Else
 t = -(Log(1 - R)) * AT + Log(q) * AT + V
 oo = t
 End If
  ARRIVETIME = ARRIVETIME + t
 xlsheet2.Cells(i, 2) = q / AT * Exp(-1 / AT * (oo - V))
 xlsheet2.Cells(i, 1) = t
 xlsheet2.Cells(i, 3) = ARRIVETIME
realar2(i) = ARRIVETIME
  Next
End Sub

Public Sub ARRIVAL3(tv) ' generate vehicles for approach 3 based on the its volume
 xlsheet3.Activate
 xlsheet3.Cells.Select
```

```
   With Selection
      xlsheet3.Cells.ClearContents
   End With
 Dim R As Double, t, AT, q, B, V As Double
Dim oo As Single
t = 0
ARRIVETIME = 0
B = 0.6
V = 2
q = Exp(-1 * B * V * tv / 3600)
AT = ((3600 / tv) - V) / q
For i = 1 To tv
Randomize
R = Rnd
Do While R = 1
Randomize
R = Rnd
 Loop
If R <= 1 - q Then
 t = V
 Else
 t = -(Log(1 - R)) * AT + Log(q) * AT + V
 oo = t
 End If
  ARRIVETIME = ARRIVETIME + t
 xlsheet3.Cells(i, 2) = q / AT * Exp(-1 / AT * (oo - V))
 xlsheet3.Cells(i, 1) = t
 xlsheet3.Cells(i, 3) = ARRIVETIME
 realar3(i) = ARRIVETIME
 Next
End Sub

Public Sub ARRIVAL4(tv) ' generate vehicles for approach 4 based on the its volume
 xlsheet4.Activate
 xlsheet4.Cells.Select
   With Selection
      xlsheet4.Cells.ClearContents
   End With

Dim R As Double, t, AT, q, B, V As Double
Dim oo As Single
t = 0
ARRIVETIME = 0
B = 0.6
V = 2
q = Exp(-1 * B * V * tv / 3600)
```

```
AT = ((3600 / tv) - V) / q
For i = 1 To tv
Randomize
R = Rnd
Do While R = 1
Randomize
R = Rnd
 Loop
 If R <= 1 - q Then
 t = V
 Else
 t = -(Log(1 - R)) * AT + Log(q) * AT + V
 oo = t
 End If
  ARRIVETIME = ARRIVETIME + t
 xlsheet4.Cells(i, 2) = q / AT * Exp(-1 / AT * (oo - V))
 xlsheet4.Cells(i, 1) = t
 xlsheet4.Cells(i, 3) = ARRIVETIME
realar4(i) = ARRIVETIME
  Next
 End Sub
Private Sub distribution()
tv = Val(sv.Text)
'TV = 600
Call ARRIVAL1(tv)
tv = Val(ev.Text)
 Call ARRIVAL2(tv)
 tv = Val(nv.Text)
 Call ARRIVAL3(tv)
 tv = Val(wv.Text)
 Call ARRIVAL4(tv)
End Sub

Private Sub min(arrive1 As Single, arrive2 As Single, arrive3 As Single, arrive4 As Single,
mintime As Single, index As Integer) ' find the vehicle which arrives at the intersection first
index = 1
mintime = arrive1
If mintime > arrive2 Then
mintime = arrive2
index = 2
End If
 If mintime > arrive3 Then
 mintime = arrive3
 index = 3
End If
 If mintime > arrive4 Then
```

```
 mintime = arrive4
 index = 4
 End If
 End Sub

Private Sub tvp(tt As Single, index, n As Integer) ' assign a vehicle’s turning movement based on the turning movement percentages
Dim sln As Single, stn As Single, srn As Single, nln As Single, ntn As Single, nrn As Single, eln As Single, etn As Single, ern As Single, wln As Single, wtn As Single, wrn As Single
sln = Val(sl.Text)
stn = Val(st.Text)
srn = Val(sr.Text)
nln = Val(nl.Text)
ntn = Val(nt.Text)
nrn = Val(nr.Text)
eln = Val(el.Text)
etn = Val(et.Text)
ern = Val(er.Text)
wln = Val(wl.Text)
wtn = Val(wt.Text)
wrn = Val(wt.Text)
If index = 1 Then
    If tt <= sln Then
    n = 1
    ElseIf tt <= sln + stn Then
    n = 2
    Else: n = 3
    End If
        ElseIf index = 2 Then
    If tt <= eln Then
    n = 1
    ElseIf tt <= eln + etn Then
    n = 2
    Else: n = 3
    End If
        ElseIf index = 3 Then
    If tt <= nln Then
    n = 1
    ElseIf tt <= nln + ntn Then
    n = 2
    Else: n = 3
    End If
        ElseIf index = 4 Then
    If tt <= wln Then
    n = 1
    ElseIf tt <= wln + wtn Then
```

```
    n = 2
    Else: n = 3
    End If
  End If
End Sub

Private Sub turning(turn, index As Integer) ' Assign numbers to a vehicle based on its turning movement and which approach it is at
Dim n As Integer
n = turn
If index = 1 Then
     If n = 1 Then
     turn = 11
          ElseIf n = 2 Then
     turn = 12
          ElseIf n = 3 Then
     turn = 13
     End If
   ElseIf index = 2 Then
    If n = 1 Then
     turn = 21
          ElseIf n = 2 Then
     turn = 22
          ElseIf n = 3 Then
     turn = 23
       End If
     ElseIf index = 3 Then
    If n = 1 Then
     turn = 31
          ElseIf n = 2 Then
     turn = 32
         ElseIf n = 3 Then
     turn = 33
       End If
      Else
     If n = 1 Then
     turn = 41
     ElseIf n = 2 Then
     turn = 42
         ElseIf n = 3 Then
     turn = 43
        End If
   End If
End Sub
```

```
Private Sub gettime(turn1 As Integer, turn2 As Integer, time As Single) ' Get the hesitation time
based on the turning movement combinaitons
Dim m As Integer
For m = 1 To 108
If turn1 = comb(m, 1) And turn2 = comb(m, 2) Then
time = comb(m, 3)
End If
 Next m
End Sub

Private Sub simulate_Click() ' Main function to run the simulation
Dim count1 As Integer, count2 As Integer, count3 As Integer, count4 As Integer
Dim totaltime As Single, totaltime1 As Single, totaltime2 As Single, totaltime3 As Single,
totaltime4 As Single
Dim time As Single, wait As Single, sum As Single, ave1 As Single, ave2 As Single, ave3 As
Single, ave4 As Single
Dim counter1 As Integer, counter2 As Integer, counter3 As Integer, counter4 As Integer, q As
Integer, counter As Integer, sameapproach As Integer
Dim arrive1 As Single, arrive2 As Single, arrive3 As Single, arrive4 As Single, mintime As
Single, smintime As Single, tt As Single
Dim j1 As Integer, j2 As Integer, j3 As Integer, j4 As Integer, index As Integer, n As Integer,
turn As Integer, turn1 As Integer, turn2 As Integer, approach As Integer
Dim q1 As Integer, q2 As Integer, k As Integer, temp As Integer, q95 As Integer, qq As Integer
Dim qlength1(1 To 2000) As Integer, qlength2(1 To 2000) As Integer, qlength3(1 To 2000) As
Integer, qlength4(1 To 2000) As Integer
For qq = 1 To 20
xlsheet1.Activate
 xlsheet2.Activate
 xlsheet3.Activate
 xlsheet4.Activate
i = 0
totaltime = 0
Call distribution
counter1 = 0
counter2 = 0
counter3 = 0
counter4 = 0
j1 = 1
j2 = 1
j3 = 1
j4 = 1
k = 1
sumservice1 = 0
sumservice2 = 0
sumservice3 = 0
sumservice4 = 0
```

```
arrive1 = xlsheet1.Cells(j1, 3)
arrive2 = xlsheet2.Cells(j2, 3)
arrive3 = xlsheet3.Cells(j3, 3)
arrive4 = xlsheet4.Cells(j4, 3)
Call min(arrive1, arrive2, arrive3, arrive4, mintime, index)
totaltime1 = arrive1
Randomize
tt = Rnd
Call tvp(tt, index, n)
turn = n
Call turning(turn, index)
turn1 = turn
approach = index
If approach = 1 Then
j1 = j1 + 1
counter1 = counter1 + 1
End If
If approach = 2 Then
j2 = j2 + 1
counter2 = counter2 + 1
End If
If approach = 3 Then
j3 = j3 + 1
counter3 = counter3 + 1
End If
If approach = 4 Then
j4 = j4 + 1
counter4 = counter4 + 1
End If
totaltime = mintime
Do While totaltime < 1800
arrive1 = realar1(j1)
arrive2 = realar2(j2)
arrive3 = realar3(j3)
arrive4 = realar4(j4)
Call min(arrive1, arrive2, arrive3, arrive4, mintime, index)

approach = index
If approach = 1 Then j1 = j1 + 1
If approach = 2 Then j2 = j2 + 1
If approach = 3 Then j3 = j3 + 1
If approach = 4 Then j4 = j4 + 1

Randomize
tt = Rnd
Call tvp(tt, index, n)
```

```
turn = n
Call turning(turn, index)
turn2 = turn
Call gettime(turn1, turn2, time)
time = time + 1
If mintime - totaltime > time Then
   totaltime = mintime + time

If approach = 1 Then  ' Simulation clock for approach1
   If index = sameapproach Then
   If mintime - totaltime1 < 3 Then
        totaltime = totaltime + 3 - (mintime - totaltime1) + time
          End If
    End If
totaltime1 = totaltime
End If

If approach = 2 Then ' Simulation clock for approach2
   If index = sameapproach Then
   If mintime - totaltime2 < 3 Then
         totaltime = totaltime + 3 - (mintime - totaltime2) + time
       End If
    End If
totaltime2 = mintime
End If

If approach = 3 Then  ' Simulation clock for approach3
   If index = sameapproach Then
   If mintime - totaltime3 < 3 Then
      totaltime = totaltime + 3 - (mintime - totaltime3) + time
       End If
    End If
totaltime3 = mintime
wait = 2 - (mintime - totaltime1) + time
End If

If approach = 4 Then  ' Simulation clock for approach4
If index = sameapproach Then
   If mintime - totaltime4 < 2.5 Then
        totaltime = mintime + 2.5 - (mintime - totaltime4) + time
       End If
    End If
totaltime4 = mintime
End If
Else
totaltime = totaltime + time
```

```
If index = 1 Then
totaltime1 = mintime
End If
If index = 2 Then
totaltime2 = mintime
End If
If index = 3 Then
totaltime3 = mintime
End If
If index = 4 Then
totaltime4 = mintime
End If
End If
turn1 = turn2
sameapproach = index

If index = 1 Then
counter1 = counter1 + 1 ' Update counter  for approach1
    If totaltime > realar1(j1) Then realar1(j1) = totaltime + 3
End If

If index = 2 Then
   counter2 = counter2 + 1 ' Update the volume counter for approach2
   If totaltime > realar2(j2) Then realar2(j2) = totaltime + 3
End If

   If index = 3 Then
counter3 = counter3 + 1 ' Update the volume counter for approach3
If totaltime > realar3(j3) Then realar3(j3) = totaltime + 3
End If

If index = 4 Then  ' Update the volume counter for approach4
counter4 = counter4 + 1
If totaltime > realar4(j4) Then realar4(j4) = totaltime + 3
End If

Call queuelength1(q, counter1, totaltime) ' Get queue length for approach1
qlength1(k) = q
Call queuelength2(q, counter2, totaltime) ' Get queue length for approach2
qlength2(k) = q
Call queuelength3(q, counter3, totaltime) ' Get queue length for approach3
qlength3(k) = q
Call queuelength4(q, counter4, totaltime) ' Get queue length for approach4
qlength4(k) = q
```

```
xlsheet4.Activate
service = totaltime – mintime ' Calculate service time for the vehicle
If index = 1 Then
wait = totaltime - xlsheet1.Cells(counter1, 3) ' Calculate control delay for the vehicle
xlsheet4.Cells(k, 25) = wait
xlsheet4.Cells(k, 20) = service
sumservice1 = sumservice1 + service
End If
If index = 2 Then
wait = totaltime - xlsheet2.Cells(counter2, 3) ' Calculate control delay for the vehicle
xlsheet4.Cells(k, 26) = wait
xlsheet4.Cells(k, 21) = service
sumservice2 = sumservice2 + service
End If
If index = 3 Then
wait = totaltime - xlsheet3.Cells(counter3, 3) ' Calculate control delay for the vehicle
xlsheet4.Cells(k, 27) = wait
xlsheet4.Cells(k, 22) = service
sumservice3 = sumservice3 + service
End If
If index = 4 Then
wait = totaltime - xlsheet4.Cells(counter4, 3) ' Calculate control delay for the vehicle
xlsheet4.Cells(k, 28) = wait
xlsheet4.Cells(k, 23) = service
sumservice4 = sumservice4 + service
End If
k = k + 1
Loop

If counter1 = 0 Or counter2 = 0 Or counter3 = 0 Or counter4 = 0 Then
qq = qq - 1
GoTo 100
End If
sum = 0
For q1 = 1 To k - 1
  sum = sum + xlsheet4.Cells(q1, 25)
  Next q1
If counter1 > 0 Then
  ave1 = sum / counter1
Else: ave1 = 0
End If
  sum = 0
For q1 = 1 To k - 1
  sum = sum + xlsheet4.Cells(q1, 26)
  Next q1
```

```
If counter2 > 0 Then
  ave2 = sum / counter2
 Else: ave2 = 0
End If
  sum = 0
For q1 = 1 To k - 1
  sum = sum + xlsheet4.Cells(q1, 27)
  Next q1
 If counter3 > 0 Then
  ave3 = sum / counter3
 Else: ave3 = 0
  End If
  sum = 0
 For q1 = 1 To k - 1
  sum = sum + xlsheet4.Cells(q1, 28)
  Next q1
  If counter4 > 0 Then
  ave4 = sum / counter4
 Else: ave4 = 0
  End If
  sum = 0

For q1 = 1 To k – 1 ' Arrange the queue lengths in ascend order
  For q2 = 1 To k
  If qlength1(q2) > qlength1(q1) Then
   temp = qlength1(q1)

   qlength1(q1) = qlength1(q2)
   qlength1(q2) = temp
   End If
   Next q2
   Next q1
For q1 = 1 To k
xlsheet4.Cells(q1, 19) = qlength1(q1)
Next q1
For q1 = 1 To k - 1
  For q2 = 1 To k
  If qlength2(q2) > qlength2(q1) Then
   temp = qlength2(q1)

   qlength2(q1) = qlength2(q2)
   qlength2(q2) = temp
   End If
   Next q2
   Next q1
For q1 = 1 To k
```

```
xlsheet4.Cells(q1, 20) = qlength2(q1)
Next q1
For q1 = 1 To k - 1
  For q2 = 1 To k
  If qlength3(q2) > qlength3(q1) Then
   temp = qlength3(q1)

   qlength3(q1) = qlength3(q2)
   qlength3(q2) = temp
   End If
   Next q2
   Next q1
For q1 = 1 To k
xlsheet4.Cells(q1, 21) = qlength3(q1)
Next q1

For q1 = 1 To k - 1
  For q2 = 1 To k
  If qlength4(q2) > qlength4(q1) Then
   temp = qlength4(q1)

   qlength4(q1) = qlength4(q2)
   qlength4(q2) = temp
   End If
   Next q2
   Next q1
For q1 = 1 To k
xlsheet4.Cells(q1, 22) = qlength4(q1)
Next q1
q95 = Round((k - 1) * 0.95, 0)
 xlsheet6.Activate
xlsheet6.Cells(qq, 35) = sumservice1 / counter1'' service time for approach1
xlsheet6.Cells(qq, 36) = sumservice2 / counter2 ' service time for approach2
xlsheet6.Cells(qq, 37) = sumservice3 / counter3' service time for approach3

xlsheet6.Cells(qq, 38) = sumservice4 / counter4 ' service time for approach4

xlsheet6.Cells(qq, 40) = "Approach1"
xlsheet6.Cells(qq, 41) = ave1 + 5.66 '  Calculate control delay for approach 1
xlsheet6.Cells(qq, 42) = qlength1(q95) ' Calculate 95th percentile queue length for approach 1


xlsheet6.Cells(qq, 43) = "Approach2"
xlsheet6.Cells(qq, 44) = ave2 + 5.66 '  Calculate control delay for approach 2
xlsheet6.Cells(qq, 45) = qlength2(q95) ' Calculate 95th percentile queue length for approach 2
```

```
xlsheet6.Cells(qq, 46) = "Approach3"
xlsheet6.Cells(qq, 47) = ave3 + 5.66  ' Calculate control delay for approach 3
xlsheet6.Cells(qq, 48) = qlength3(q95) ' Calculate 95th percentile queue length for approach 3

xlsheet6.Cells(qq, 49) = "Approach4"
xlsheet6.Cells(qq, 50) = ave4 + 5.66 ' Calculate control delay for approach 4
xlsheet6.Cells(qq, 51) = qlength4(q95) ' Calculate 95th percentile queue length for approach 4

sum1 = sum1 + ave1 + 5.66
sum2 = sum2 + qlength1(q95)

sum3 = sum3 + ave2 + 5.66

sum4 = sum4 + qlength2(q95)

sum5 = sum5 + ave3 + 5.66
sum6 = sum6 + qlength3(q95)
sum7 = sum7 + ave4 + 5.66
sum8 = sum8 + qlength4(q95)
100  Next qq


output.Print "Approach1", " released traffic", counter1, totaltime ' outputs in the output window
output.Print " delay", sum1 / 20, " queue length", sum2 / 20
output.Print "Approach2", " released traffic", counter2
output.Print " delay", sum3 / 20, " queue length", sum4 / 20
output.Print "Approach3", " released traffic", counter3
output.Print " delay", sum5 / 20, " queue length", sum6 / 20
output.Print "Approach4", " released traffic", counter4
output.Print " delay", sum7 / 20, " queue length", sum8 / 20
End Sub

' Calculate queue length for approach1
Private Sub queuelength1(q As Integer, counter1 As Integer, totaltime As Single)
Dim i As Integer
If counter1 = 0 Then
q = 0
Else
i = counter1 + 1
q = 0
Do While totaltime > xlsheet1.Cells(i, 3) And xlsheet1.Cells(i, 3) > 0
i = i + 1
q = q + 1
Loop
End If
End Sub
```

```
' Calculate queue length for approach1
Private Sub queuelength2(q As Integer, counter2 As Integer, totaltime As Single)
Dim i As Integer
If counter2 = 0 Then
q = 0
Else
i = counter2 + 1
q = 0
Do While totaltime > xlsheet2.Cells(i, 3) And xlsheet2.Cells(i, 3) > 0
i = i + 1
q = q + 1
Loop
End If
End Sub

' Calculate queue length for approach3
Private Sub queuelength3(q As Integer, counter3 As Integer, totaltime As Single)
Dim i As Integer
If counter3 = 0 Then
q = 0
Else
i = counter3 + 1
q = 0
Do While totaltime > xlsheet3.Cells(i, 3) And xlsheet3.Cells(i, 3) > 0
i = i + 1
q = q + 1
Loop
End If
End Sub

' Calculate queue length for approach4
Private Sub queuelength4(q As Integer, counter4 As Integer, totaltime As Single)
Dim i As Integer
If counter4 = 0 Then
q = 0
Else
i = counter4 + 1
q = 0
Do While totaltime > xlsheet4.Cells(i, 3) And xlsheet4.Cells(i, 3) > 0
i = i + 1
q = q + 1
Loop
End If
End Sub
```

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